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ERRATA.

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Page 356, line 6, in the paper "Pressure, Resistance, and Stability of Earth," by J. C. Meem, M. Am. Soc. C. E. Instead of the equation given, insert the following:

$$W_1 = \frac{l \times \frac{l}{2} \tan. \alpha \times W}{2} = \frac{l^2}{4} \left\{ \frac{1}{2} (90° - \phi) + \phi \right\} W$$

$$= \frac{l^2}{4} \tan. \left(45° + \frac{\phi}{2} \right) W.$$

Transactions, Vol. LXXIV.

Page 95, lines 12 and 31: For "Hatton" read "Hetton."

Page 176, line 12: For "numbers" read "members."

Page 357, line 17 from the bottom: For "W. B. Smith" read "F. D. Smith."

AMERICAN SOCIETY OF CIVIL ENGINEERS
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TRANSACTIONS

Paper No. 1203

STEEL CENTERING USED IN THE CONSTRUCTION OF THE ROCKY RIVER BRIDGE, CLEVELAND, OHIO.*

By Wilbur J. Watson, M. Am. Soc. C. E.

With Discussion by Messrs. L. J. Mensch and Wilbur J. Watson.

There has recently been completed, near Cleveland, Ohio, a concrete arch bridge having a clear main span of 280 ft. and a rise of 80 ft. The length of the structure over all is 708 ft. The roadway is 40 ft. wide, and is flanked by two 8-ft. footwalks. The structure was built by Cuyahoga County, over Rocky River, on Detroit Avenue, and connects the suburban villages of Lakewood and Rocky River. It will carry a double-track interurban railroad, and heavy vehicular traffic.

The general design is quite similar to that of the Walnut Lane Bridge, at Philadelphia, but a radical departure from the usual methods was made in the construction, as steel, instead of timber centering, was used for the main arch.

The contract for this bridge was let on August 22d, 1908, and prior to that time, a contracting firm, desiring to submit a bid, retained the writer to design and estimate the cost of the centering and forms required.

The writer made preliminary plans for both timber and steel centers, including several types of the latter, and finally decided on the use of the three-hinged arch as being the most practicable and economical. Among the advantages which the steel centers possessed were the following:

Presented at the meeting of June 7th, 1911.

1st.—Economy: These centers were estimated to be at least $15 000 cheaper than timber centering of the usual type.

2d.—Freedom from danger of being carried out by ice. It was evident that these centers would have to be in the river at least one winter, and there was great danger of losing them if timber centering of the ordinary type were used.

3d.—The three-hinged arch is a statically determinate structure, and it was a simple matter to compute the exact stresses and deflections for each position of the load.

4th.—The deflection of the hinged arch under load was found to be far less than that of centers of the truss or beam type.

As the use of steel centers for the construction of permanent masonry arches of such long span had never been tried before, the latter was an essential consideration. As far as the writer is aware, these were the first steel centers to be used for any but very short spans.

The steel centers used in the construction of the 150-ft. arches of the Delaware River Bridge of the Delaware, Lackawanna and Western Railway were designed some months later, when the centers described herein were well under way in the shop.

After the contract was awarded, the writer was instructed to prepare a complete design for these centers.

In computing the stresses, it was assumed that only the radial component of the weight of each voussoir was to be carried by the steel centers, and that the tangential component would be carried by the timber bows, or by reinforced concrete struts placed in the key sections.

In carrying out the work, both of these provisions were made, the bows being of sufficient strength to carry the entire tangential components of the loads, and reinforced concrete struts being also provided, as indicated on the stress diagram, Plate I.

This assumption has been criticized, and the best proof of its accuracy which the writer can offer is the fact that the observed and computed deflections corresponded precisely, and they would not have done so had not this assumption as to the distribution of the load been correct. The bows were simply laid on the cross-beams, being left free to move longitudinally thereon, and were carefully framed with butt joints, so as to act as arch ribs.

PLATE I.
TRANS. AM. SOC. CIV. ENGRS.
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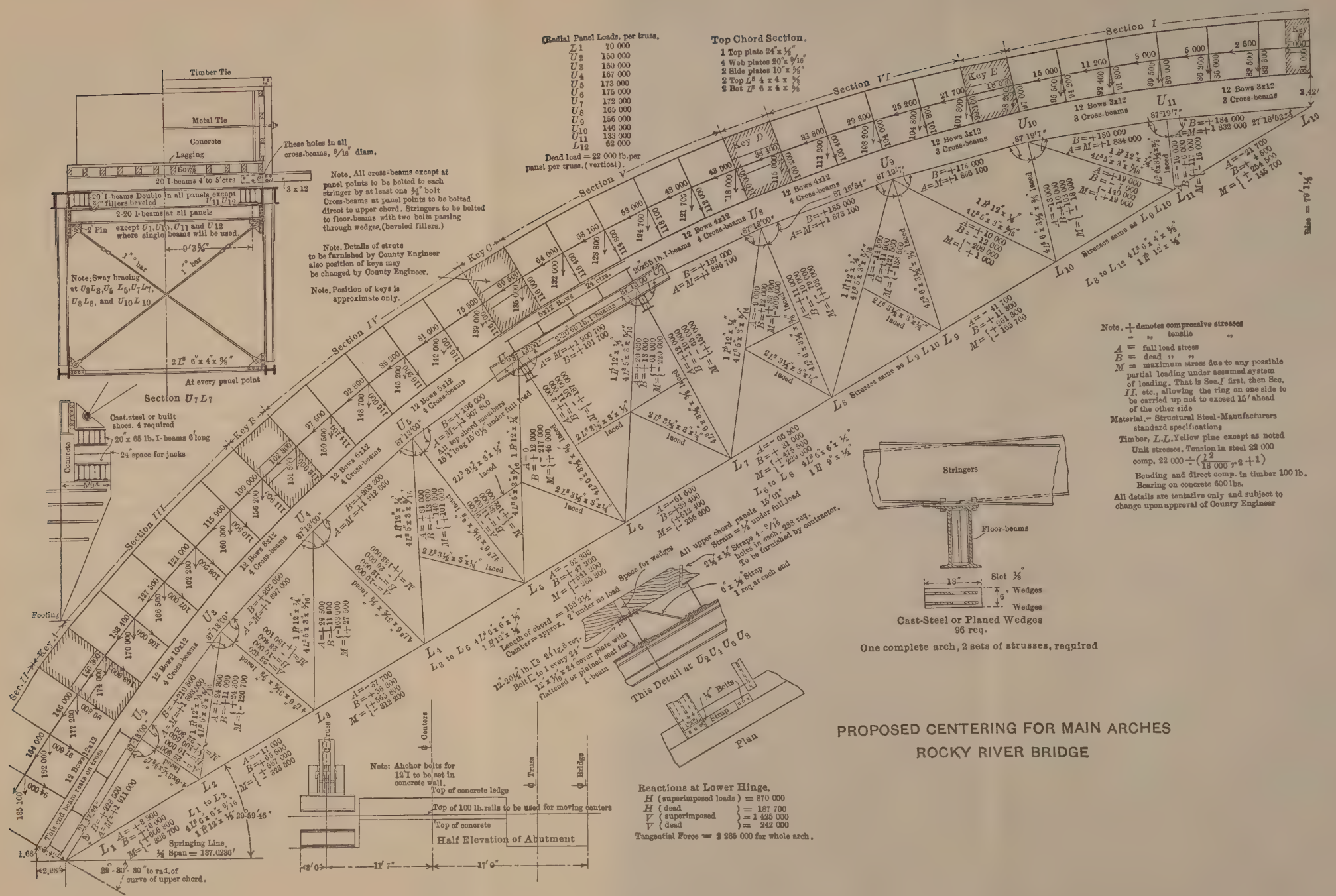
WATSON ON
STEEL CENTERING FOR CONCRETE BRIDGE.


Radial Panel Loads, per truss.
L1 70 000
U2 150 000
U3 160 000
U4 167 000
U5 173 000
U6 175 000
U7 172 000
U8 165 000
U9 156 000
U10 146 000
U11 133 000
L12 62 000
Dead load = 22 000 lb. per panel per truss. (vertical).
Top Chord Section.
1 Top plate 24" x ½"
4 Web plates 20" x 9/16"
2 Side plates 10" x ⅜"
2 Top L 4 x 4 x ⅜
2 Bot L 6 x 4 x ⅝
Section I
Section II
Section III
Section IV
Section V
Section VI
Section U7L7
Timber Tie
Metal Tie
Concrete
Lagging
Bows
20 I-beams 4' to 5' ctrs
20 I-beams Double in all panels except U11 U12
2-20 I-beams at all panels
2 Pin
except U1,U10, U11 and U12 where single beams will be used.
1" ° bar
Note; Sway bracing at U2L2, U5 L5, U7L7, U8L8, and U10 L10
2L 6" x 4" x ⅝"
At every panel point
These holes in all cross-beams, 9/16" diam.
3 x 12
Note. All cross-beams except at panel points to be bolted to each stringer by at least one ¾" bolt
Cross-beams at panel points to be bolted direct to upper chord. Stringers to be bolted to floor-beams with two bolts passing through wedges. (beveled fillers.)
Note. Details of struts to be furnished by County Engineer also position of keys may be changed by County Engineer.
Note. Position of keys is approximate only.
Key A
Key B
Key C
Key D
Key E
Key F
12 Bows 5x12
4 Cross-beams
12 Bows 4x12
12 Bows 3x12
3 Cross-beams
12 Bows 8x12
3 Cross-beams
2-20 65 lb. I-beams
Cast-steel or built shoes. 4 required
20" x 65 lb. I-beams 6' long
24" space for jacks
Concrete
Footing
Springing Line
⅛ Span = 137.0235'
29° 30' 30" to rad. of curve of upper chord.
Stringers
Floor-beams
Slot ⅞"
Wedges
Cast-Steel or Planed Wedges
96 req.
One complete arch, 2 sets of strusses, required
Note. + denotes compressive stresses
− " tensile "
A = full load stress
B = dead " "
M = maximum stress due to any possible partial loading under assumed system of loading. That is Sec. I first, then Sec. II, etc., allowing the ring on one side to be carried up not to exceed 15° ahead of the other side
Material.– Structural Steel-Manufacturers standard specifications
Timber, L.L. Yellow pine except as noted
Unit stresses. Tension in steel 22 000
comp. 22 000 ÷ (l²/18 000 r² + 1)
Bending and direct comp. in timber 100 lb.
Bearing on concrete 600 lbs.
All details are tentative only and subject to change upon approval of County Engineer
All upper chord panels 15'01"
Strain = ⅛ under full load
Strings 4 - 9/16" holes in each. 288 req.
To be furnished by contractor.
Span for wedges
2 under no load
Length of chord = 156'6¼"
Camber = approx. 2'
Bolt L to I every 24"
12" x ⅞" x 24" cover plate with flattened or planed seat for I-beam
This Detail at U2 U4 U6 U8
Plan
Truss
Centers
Bridge
Note: Anchor bolts for 12" I to be set in concrete wall.
Top of concrete ledge
Top of 100 lb. rails to be used for moving centers
Top of concrete
Half Elevation of Abutment
Reactions at Lower Hinge.
H (superimposed loads) = 870 000
H (dead) = 187 700
V (superimposed) = 1 425 000
V (dead) = 242 000
Tangential Force = 2 285 000 for whole arch.
Rise = 77' 1¼"

PROPOSED CENTERING FOR MAIN ARCHES
ROCKY RIVER BRIDGE

The design was computed on the assumption that the arch ring would be concreted in 15-ft. transverse sections, or voussoirs, and it was assumed that a section on one side of the arch might be completed in advance of the corresponding section on the other side. It was also assumed that the placing of the voussoirs might take place in any order, symmetrically about the center. Before checking, however, the order of loading had been determined, and the computations were based on the assumption of such order of loading.

A unit stress of 21 000 lb. per sq. in. on medium open-hearth steel was used, making the proper reduction for columns according to Gordon's formula for pin-connected columns.

It was decided to use bolts instead of rivets for all field connections, in order to facilitate removal, after the completion of the work.

These bolts were turned to a diameter of $\frac{29}{32}$ in., and placed in $\frac{15}{16}$-in. reamed holes. All holes for field connections were reamed in the field, using pneumatic reamers.

The centers were designed to be erected in an approximately vertical position, and then lowered into position from the banks. The contractor, however, preferred to erect them on light timber falsework, which was to be removed as soon as the trusses were in place. An 800-ft. cableway, installed for handling the concrete, was also utilized in erecting the centers, and in their removal.

The weight of steel in these centers was approximately 400 tons, of which about 160 tons were steel I-beams which were used in the completed structure, being placed across the opening between the two arch ribs, transversely to the axis of the bridge, and encased in concrete.

The use of so large a part of the steel in the permanent structure was quite an item contributing to the remarkable economy of these centers. The cost was approximately as follows:

Material and fabrication	$20 000	
Freight and erection	5 500	
Dismantling	1 200	
Total		$26 700
Value of steel I-beams used in permanent structure	$7 000	
Value of remainder of structure as scrap material	2 500	
		9 500
Net cost of centers		$17 200

The foregoing figures, exclusive of wedges, take no account of overhead or engineering charges, nor the cost of timber lagging and moving the centers, the latter being a very small item.

The two most troublesome problems to be solved were the provision for motion of the steel arches, due to temperature changes, and the design of the lowering jacks.

The County Engineer's specifications provided that the timber centering contemplated therein should be kept wet during the construction of the keys, and the writer's original intention was to keep the steel centers wet during the concreting and setting up of the last key (at the crown). As city water would be used, and at a practically constant temperature as it emerged from the pipes, it would seem that by this means, and by using canvas to screen the steel from the direct rays of the sun, the position of the crown hinge might be kept at a constant elevation for the length of time necessary for the concrete in the key to harden.

It was decided, however, that this was unnecessary, as the arch was not reinforced, and any rise of the center hinge would only serve to open up the voussoir joint at the crown.

This voussoir, therefore, was concreted on a cool day, and the surfaces of the two adjacent arch sections were oiled in order to prevent an immediate bond, so that there would be no danger of the key concrete being prematurely loaded; and the joint at the crown was allowed to open and close with temperature changes. The opening was as great as $\frac{5}{16}$ in. at one time. The rise and fall of the center hinge was so sensitive to changes of temperature, that Mr. Stevens, the Resident Engineer, states that he could determine the temperature of the air by measuring the opening.

The computed deflection of each truss, or half arch, under full load was 2 in., and the trusses were cambered this amount. The observed deflection corresponded to this precisely. It is evident that, in the case of a reinforced arch, it would have been necessary to carry out the original intention of wetting down the centers, but it is difficult to see how the principle of reinforcing an arch of this kind can be defended, because the live load is practically negligible in comparison with the dead load, and the curve of the arch can be adapted to the curve of the equilibrium polygon, thus practically eliminating all bending and shearing forces in the arch ring.

FIG. 1.—STEEL CENTERS FOR ROCKY RIVER BRIDGE.

FIG. 2.—APPROACH SPANS, AND STEEL CENTERS, ROCKY RIVER BRIDGE.

Plate I shows the stress sheet for one-half of one center, and details of the bows, lagging, etc. It will be noted that the arch ring is divided into voussoirs, which are given Roman numerals corresponding to the sequence of concreting. The keys are designated by letters, also corresponding to the sequence in which they were concreted.

It was found by experiment that a concrete composed of one part Portland cement, one part sand, and two parts stone would attain as great a strength in 7 days as the concrete used in the arch ring would attain in 30 days. The rich mix, therefore, was used in the keys, in order that the centers might be the sooner removed. Table 1 gives the results of these tests on concrete.

TABLE 1.—Crushing Strength of Six-Inch Concrete Cubes; Rocky River Bridge.

Kind of concrete.	Age.	Crushed.	Cracked.	Kind of concrete.	Age.	Crushed.	Cracked.
1:3:5	30 days.	1 718	1 368	1:3:5	6 months.	2 781	2 509
"	" "	2 045	1 527	"	" "	2 727	2 454
"	" "	2 018	1 418	"	" "	2 127	1 800
"	" "	1 936	1 636	"	" "	1 909	1 527
"	" "	1 909	1 527				
"	" "	2 454	2 072	1:2:4	30 days.	2 781	2 127
"	" "	2 182	2 018	"	" "	2 836	2 836
"	" "	2 509	2 454	"	" "	3 270	2 181
"	" "	2 073	1 265	"	" "	3 380	770
"	" "	2 509	1 582	"	" "	3 272	1 910
"	" "	2 564	1 582				
"	" "	1 854	1 091	"	90 "	2 672	2 180
				"	" "	2 836	1 910
"	90 "	2 563	2 127	1:1:2	7 "	2 945	1 091
"	" "	2 509	1 964	"	" "	3 436	2 727
"	" "	2 127	1 361		Granolithic Facing.		
"	" "	1 936	1 909				
"	" "	2 891	1 909	1:1:2	90 days.	2 727	2 727
"	" "	2 836	1 963	"	" "	2 945	2 836

Tables 2 and 3 give the observed deflections of the steel arch under different conditions of loading.

The rise and fall of the centers due to changes of temperature were very carefully noted, both before and after loading, as were also the deflections due to partial loadings.

The first concrete in the main arch rib was placed on August 6th, 1909, and the voussoirs were completed August 30th. The keys were run on September 3d to 9th, and the centers were struck on September 28th. When the centers were moved into position for the second rib, the first arch rib and the columns, up to the springing line of the spandrel arches, were in place.

TABLE 2.—Elevations on Alternate Points of Each Half Arch Taken During Placing of Concrete; Rocky River Bridge.

East Side.

Date.	Ct. Pin.	No. 1.	No. 2.	No. 3.	No. 4.	No. 5.	Temp.	Remarks.
8/ 6/09	76.819	74.540	66.150	52.315	33.495	10.415	84°	25 tons in place.
8/ 9/09	76.708	74.470	66.150	52.330	33.505	10.410	85°	465 tons in place, Sec. I.
8/11/09	76.683	74.458	66.123	52.368	33.508	10.408	74°	Sec. I and 6 ft. of Sec. II in place.
8/13/09	76.680	74.468	66.113	52.313	33.508	10.403	80°	Sec. II in place.
8/17/09	76.682	74.466	66.106	52.296	33.486	10.391	77°	¾ Sec. III in place.
8/19/09	76.699	74.544	66.109	52.294	33.464	10.384	78°	Sec. III in place.
8/24/09	76.762	74.482	66.102	52.262	33.422	10.367	86°	¾ Sec. IV in place.
8/26/09	76.782	74.493	66.103	52.233	33.408	10.368	85°	Sec. IV in place.
8/28/09	76.773	74.463	66.038	52.188	33.388	10.358	73°	Sec. V and ½ VI in place.
8/31/09	76.777	74.426	66.006	52.166	33.381	10.351	80°	Voussoirs in place.

West Side.

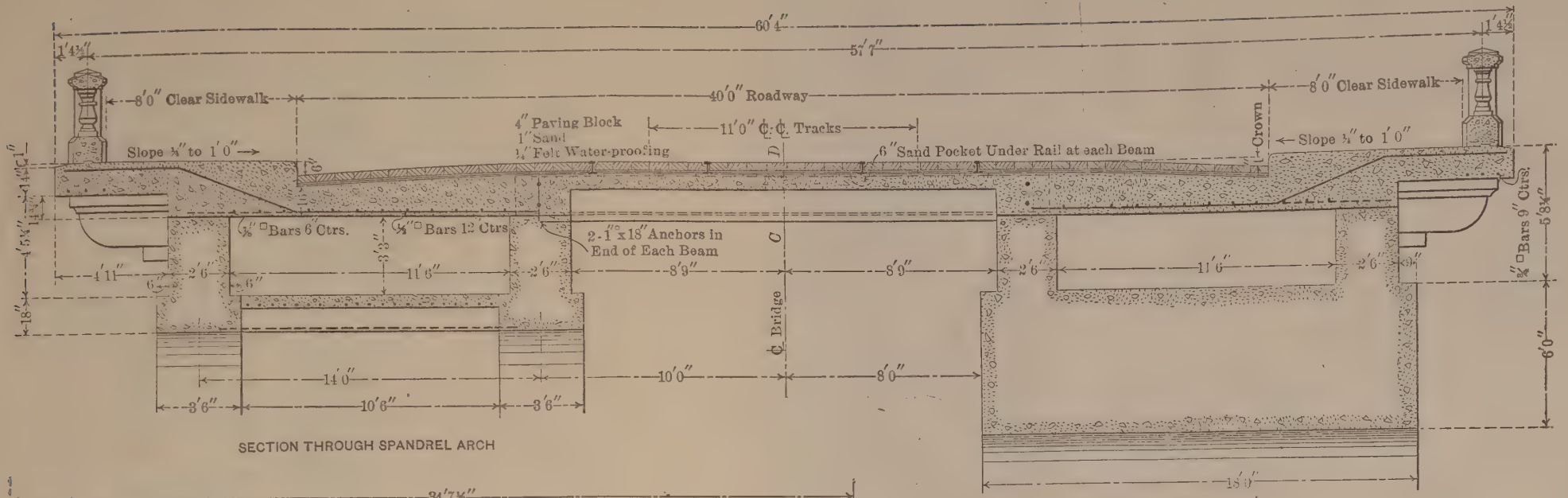
8/ 6/09	.....	74.533	66.128	52.328	33.483	10.393	84°	25 tons in place.
8/ 9/09	.....	74.455	66.210	52.310	33.490	10.400	85°	465 tons in place. Sec. I.
8/11/09	.....	74.428	66.098	52.288	33.478	10.390	74°	Sec. I and about 6 ft. of Sec. II in place.
8/13/09	.....	74.444	66.181	52.301	33.491	10.396	80°	Sec. II in place.
8/17/09	.....	74.429	66.169	52.289	33.469	10.381	77°	¾ Sec III in place.
8/19/09	.....	74.461	66.161	52.281	33.456	10.376	78°	Sec. III in place.
8/24/09	.....	74.463	66.078	52.243	33.408	10.363	86°	¾ Sec. IV in place.
8/26/09	.....	74.453	66.058	52.213	33.388	10.353	85°	Sec. IV finished.
8/28/09	.....	74.408	65.988	52.178	33.378	10.348	73°	Sec. V and ½ Sec. VI in place.
8/31/09	.....	74.391	66.011	52.171	33.371	10.346	80°	Voussoirs in place.

The unit stress in the arch rib due to its own weight is about 245 lb. per sq. in. The settlement of the crown when the centers were removed amounted to 0.038 ft. The curve of the intrados, as constructed, varies from the computed curve less than ½ in. at the point of maximum variation.

It was at first intended to use hydraulic jacks for lowering the centers, and the writer, in connection with the Duff Manufacturing Company, of Pittsburg, worked out a plan for lowering, using 32 jacks, in nests of four, each jack having a lowering capacity of 105 tons; nests of eight were to be operated by a single pump and reservoir.

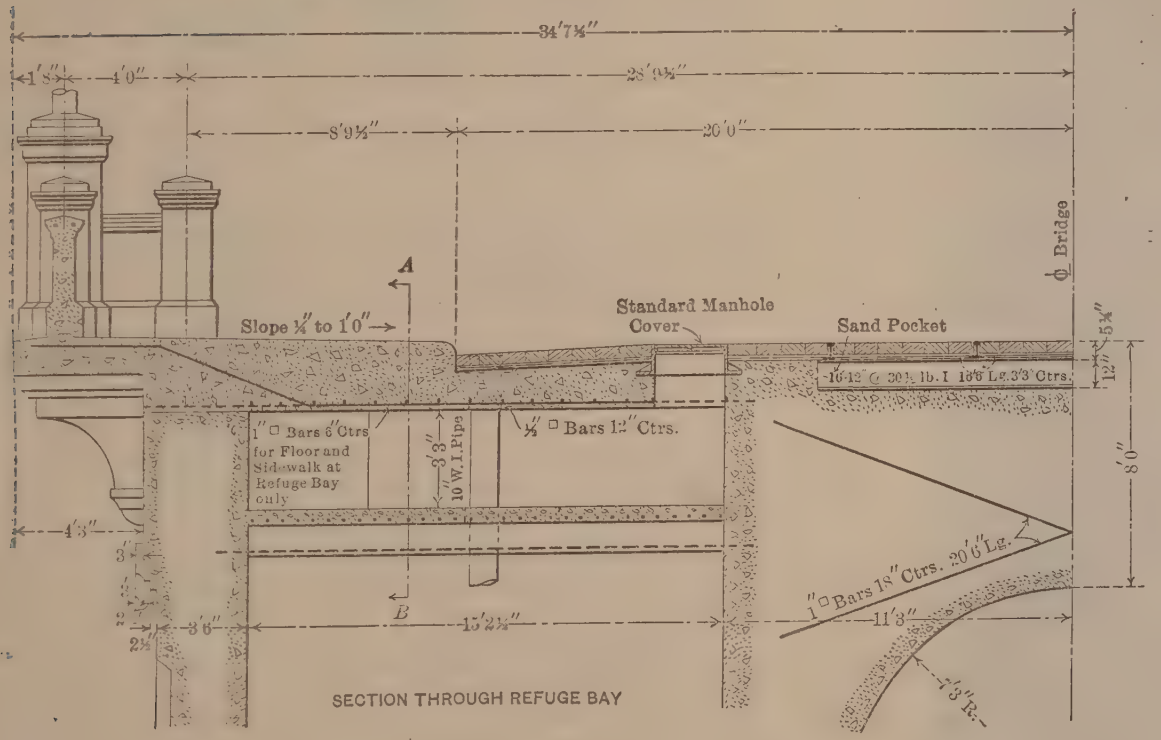
As it would be necessary to lock these jacks, owing to the length of time that they would be called on to sustain their load in one position, the Duff Manufacturing Company, designed a special locking device which could be released without lifting the ram.

In addition to the hydraulic jacks, the use of sand-boxes, screw-wedges, and screw-jacks was investigated. It was found that the sand-boxes were a little cheaper than any of the other devices, but, owing

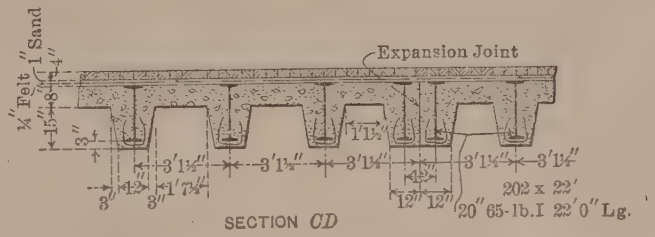


SECTION THROUGH SPANDREL ARCH

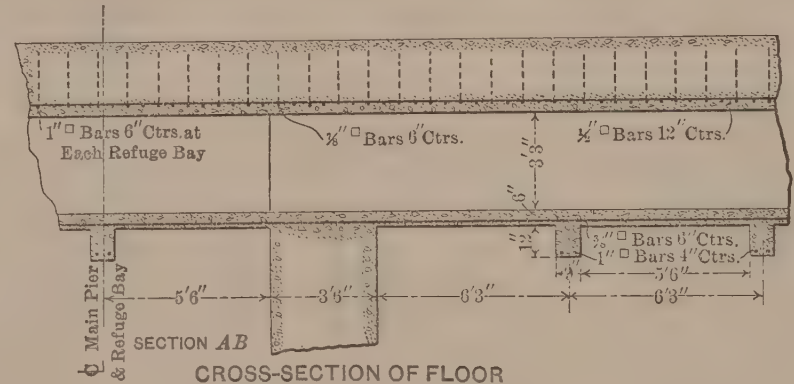
SECTION THROUGH 280′ ARCH



SECTION THROUGH REFUGE BAY

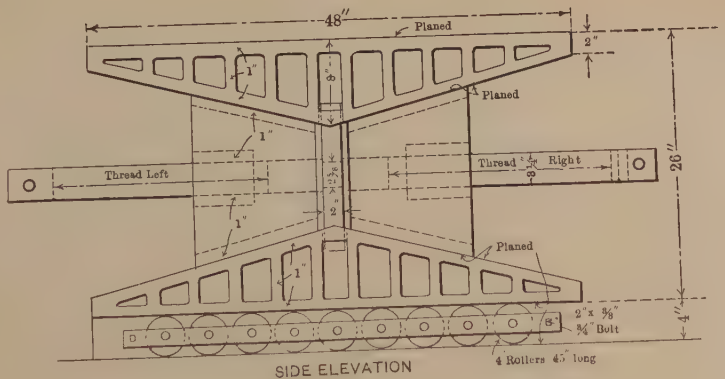


SECTION *CD*

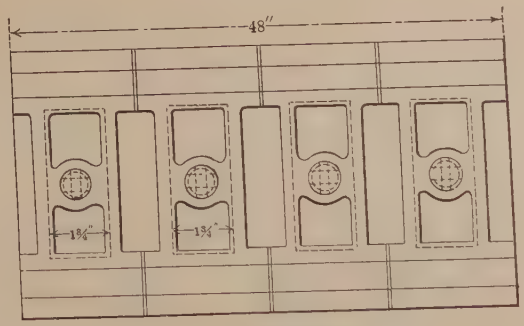


SECTION *AB*

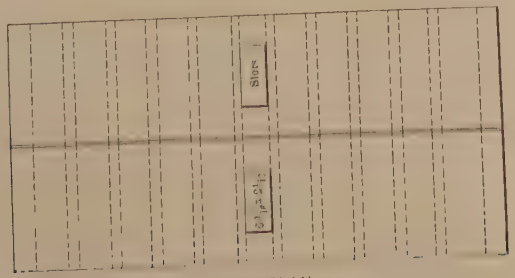
CROSS-SECTION OF FLOOR
DETROIT AVENUE VIADUCT.
OVER ROCKY RIVER.



SIDE ELEVATION



END ELEVATION



HALF PLAN

LOWERING WEDGES
FOR CENTERS OF
ROCKY RIVER BRIDGE.

Material: Cast steel, except rods and rollers, annealed.

FIG. 1.

to the fact that these boxes would be immersed in water (as their position was below mean water level), it was thought that it would be practically impossible to keep the sand dry. The sand-boxes, as contemplated, were to be of cast steel, and the estimated cost was but little less than that of the double-acting screw-wedges, which were finally adopted.

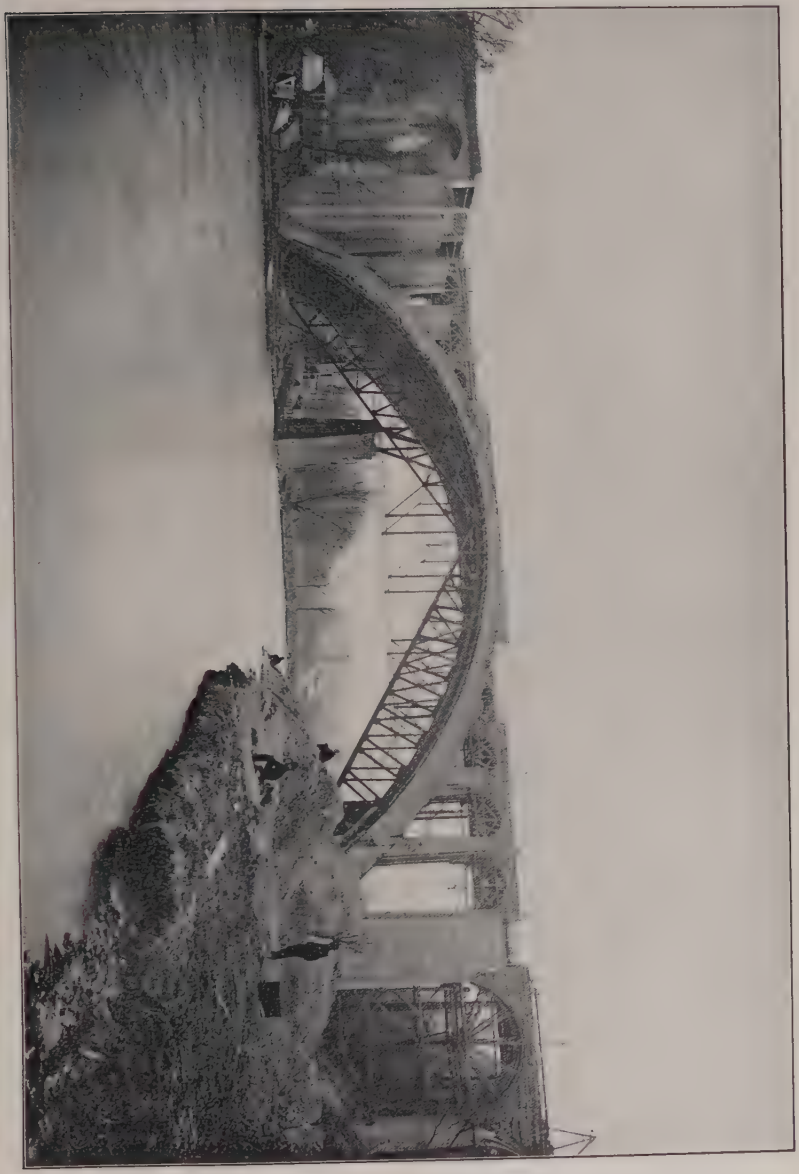
TABLE 3.—Elevations of South Center Pin Due to Changes of Temperature After Concrete Was Placed; Rocky River Bridge.

Date.	Hour.	Elev.	Temperature.		Remarks.
			In shade.	In sun.	
9/ 9/09	1.00 P. M.	76.771	75°	.....	Cloudy.
9/ 9/09	5.35 P. M.	76.758	68°	.....	Raining. Key finished at 7 P. M. Temp., 65°.
9/10/09	8.30 A. M.	76.743	69°	.....	Trans. reading same as 7.00 last night when key finished.
9/10/09	12.30 P. M.	76.762	74°	78°	Trans. reading same as last night and this A. M. Sun since 10.30.
9/10/09	3.00 P. M.	76.761	75°	.....	Clear. Trans. reading same as last night and when key completed.
9/11/09	8.45 A. M.	76.754	69°	81°	Clear. Trans. reading same as last night and when key completed.
9/11/09	11.30 A. M.	76.754	71°	78°	Clear. Trans. reading same as last night and when key completed.
9/13/09	8.40 A. M.	76.770	79°	86°	Clear. Joint open W. side of key, $\frac{1}{16}$ in.
9/13/09	1.00 P. M.	76.785	85°	96°	" " " $\frac{1}{4}$ in.
9/13/09	4.00 P. M.	76.783	85°	87°	" " " $\frac{5}{16}$ "
9/14/09	8.55 A. M.	76.770	84°	100°	" " " $\frac{1}{16}$ "
9/14/09	1.45 P. M.	76.786	85°	93°	" " " $\frac{1}{4}$ "
9/14/09	4.10 P. M.	76.782	82°	85°	" " " $\frac{3}{16}$ "
9/15/09	9.00 A. M.	76.764	75°	78°	" " " $\frac{1}{4}$ "
9/15/09	12.45 P. M.	76.764	81°	.....	Cloudy. " " $\frac{1}{16}$ "
9/15/09	4.00 P. M.	76.755	69°	.....	" " " $\frac{3}{16}$ "
9/16/09	9.00 A. M.	76.749	69°	.....	" " " $\frac{1}{16}$ "
9/16/09	1.00 P. M.	76.752	67°	78°	Joint open, $\frac{1}{16}$ in.
9/16/09	4.00 P. M.	76.754	67°	75°	" " $\frac{1}{32}$ "
9/17/09	9.00 A. M.	76.739	69°	78°	" " 0 "
9/17/09	1.05 P. M.	76.759	71°	82°	" " 0 "
9/17/09	4.00 P. M.	76.758	71°	80°	" " $\frac{1}{16}$ "
9/18/09	8.55 A. M.	76.748	67°	85°	" " 0 "
9/18/09	11.50 A. M.	76.751	67°	83°	" " $\frac{1}{32}$ "
9/20/09	8.50 A. M.	76.750	77°	82°	" " $\frac{1}{16}$ "
9/20/09	1.00 P. M.	76.780	83°	86°	" " $\frac{1}{8}$ "
9/20/09	4.00 P. M.	76.780	80°	85°	" " $\frac{1}{8}$ "

The design of these double-acting steel wedges, as finally used, Fig. 1, was suggested by A. M. Felgate, County Bridge Engineer, while looking at the writer's design for a similar single-acting screw-wedge, and appeared to be exactly what was needed. They were made of carefully annealed cast steel, in four sections, planed on all sliding and bearing surfaces, and very carefully adjusted to uniform height when set in position. The motion allowed for was a vertical drop of $6\frac{1}{2}$ in., and the slopes were calculated so that locking would not be required to hold them under full load. They were lowered by four men on each screw. Each wedge weighed approximately 1 350 lb.

FIG. 1.—CONCRETING ARCH RING, ROCKY RIVER BRIDGE.

FIG. 2.—MOVING CENTERING TO POSITION UNDER SECOND RIB.



CENTERING, AND CONCRETE ARCH, ROCKY RIVER BRIDGE.

Next in cost to the steel wedges were the screw-jacks, each of which was designed to lower 105 tons and lift 22 tons. The hydraulic jacks were found to be the most expensive.

Estimated cost of sand-boxes	$1 300
" " " screw-wedges	1 650
" " " screw-jacks	2 000
" " " hydraulic jacks	2 400

Particular attention is called to the high class of shop workmanship required on the centers. The reaming was done with unusual care. The planed joints were required to be perfect, and were the best the writer has ever seen. In fact, the workmanship was better than that required for standard railroad bridge work. In view of this, and also of the fact that the structure was free from impact, the writer is of the opinion that the unit stresses assumed were not unreasonable.

The bridge was built by Cuyahoga County, under the direction of A. B. Lea, former County Engineer, and F. R. Lander, present County Engineer. The structure was designed by A. M. Felgate, County Bridge Engineer. The Contractor was the Schillinger Brothers Company. The Resident Engineer was A. L. Stevens. The centers were built by the Interstate Engineering Company, and the wedges were cast by the Variety Iron Works Company, of Cleveland.

DISCUSSION

Mr. Mensch.

L. J. MENSCH, M. AM. SOC. C. E. (by letter).—The three-hinged arch centering described by Mr. Watson is an interesting form of scientific centering for concrete construction. In 1898 the writer designed three-hinged steel arches of 51-ft. span for the arching of the River Wien at Vienna, Austria.* About 100 arches were used, as they were cheaper than timber centering on account of the high price of timber. These arches were figured for the radial component of the load only, just as Mr. Watson's were, and the deflection was obtained by the graphical method, according to Williot. The writer seriously doubts, however, whether the steel centering used for the Cleveland bridge was cheaper than a scientifically designed timber centering would have been. The steel centering simply replaced the wooden supports holding the timber lagging in place, and it would hardly have taken more than 400 000 ft. b. m. for the necessary supports, bracing, and pilings, for both portions of the arch.

From the paper it appears that the cost of the steel centering, after allowing for the steel I-beams and scrap material, was $17 200, which with incidentals was easily brought up to $20 000. The cost of 400 000 ft. of timber, erected and dismantled, would have been considerably less than $20 000, especially as there was time to wait for the shipment of lumber from the South, and considering that the value of the old material to the contractor would have been a large percentage of the original cost.

In 1906 steel centering was used for a reinforced concrete bridge of 140 ft. span and 23 ft. rise, at Deurne-Merxem, France. It consisted of an iron trestle with steel supports about 8 ft. apart in the direction of the bridge and 6 ft. apart crossways, leaving a clear waterway, 32 ft. wide, in the center, which was spanned by an arched box section consisting of two 12-in. channels and 10 by $\frac{1}{4}$-in. cover-plates. The supports consisted of two angle irons varying from 2 by 2 by $\frac{1}{4}$ in. to $2\frac{1}{2}$ by $2\frac{1}{2}$ by $\frac{5}{16}$ in.

It would be interesting to learn how the shrinkage of the rich concrete in Key *A* was overcome, so that the contact of the surface with the next higher voussoir was preserved in every point.

The writer is rather surprised at the author's statement that reinforced concrete arches of spans like that of the Cleveland bridge are impracticable,

"because the live load is practically negligible in comparison with the dead load, and the curve of the arch can be adapted to the curve of the equilibrium polygon, thus practically eliminating all bending and shearing forces in the arch ring."

An arch bridge can be shaped according to an equilibrium polygon of only one system of loading. Any other system of loading produces

* *Engineering* (London), Vol. LXVIII, p. 325 (Sept. 15th, 1899).

bending moments and shearing forces in every point of the arch, and in order to keep these stresses within allowable limits in a solid concrete or masonry arch, the thickness of the arch must be increased beyond economical sizes. Mr. Mensch.

Compare the Cleveland bridge with the design for a reinforced concrete bridge, with a span of 300 ft. and a rise of 90 ft., made by the writer.* The average uniform total dead and live load for which the thrust is to be figured, is given as 3 233 lb. for 6 sq. ft., or the thrust for a span of 280 ft. and a rise of 80 ft. and for a width of 6 ft. of the bridge $= 3\,233 \times \frac{280 \times 280}{8 \times 80} = 396\,000$ lb.

It is also shown† that the greatest moment in an arch of the shape proposed, from a moving uniform load, occurs at a point distant three-tenths of the span from the abutment, and $= \frac{pl^2}{100}$ when p equals the uniform live load per linear foot and l the span in feet. This value is about 12% greater than that commonly found, because ordinarily it is assumed that the greatest moment occurs when exactly half of the span is uniformly loaded, while in reality it is produced when the load covers only $\frac{44}{100}$ of the span. For a 6-ft. width of bridge, $p = 600$ lb. and $l = 280$ ft., and the maximum moment $= \frac{600 \times 280 \times 280}{100} =$ 470 400 ft-lb.

The section, shown by Fig. 3, has an area equivalent to 1 915 sq. in. and a moment of resistance of 17 700 cu. in.; hence the uniform stress produced by the thrust is very closely $\frac{396\,000}{1\,915} = 214$ lb. per sq. in., and the extreme tensile and compressive stresses in the concrete $= \frac{470\,400}{17\,700} = 266$ lb. per sq. in., or the maximum compressive stress $= 214 + 266 = 480$ lb. per sq. in., and the maximum tensile stress $= 266 - 214 = 52$ lb. per sq. in.

All ordinances and safe practice require that tensile stresses in the concrete must be taken care of entirely by the steel reinforcement. In order to satisfy the most timid engineer, it will be assumed that the entire bending moment must be taken up by the reinforcement, neglecting the favorable influence of the thrust of the arch. The area of seven 1½-in. rods is 12.355 sq. in., the distance from center to center of reinforcement is 40 in. or 3.33 ft., and, therefore, the stress in the steel rods from the entire bending moment $= \frac{470\,400}{3.33 \times 12.355} = 11\,450$ lb. per sq. in., which is certainly a conservative figure, if this stress did exist.

The writer estimates that the greatest tensile stress in the steel rods in this case does not exceed 1 400 lb. per sq. in., and the greatest

* "The Reinforced Concrete Pocket Book," p. 178.

† *Ibid*, p. 158.

Mr. Mensch.

compressive stress does not exceed 7 000 lb. per sq. in. The temperature stresses for the change of 50° Fahr., do not amount to more than 25 lb. per sq. in. and, therefore, are negligible.

This calculation assumes that the section shown by Fig. 3 is at the three-tenths point of the arch. The writer has advised that this section be used as the crown section, gradually increasing it 50% until it reaches the abutment, so that the actual stresses are easily 20% smaller than those already given.

The weight of the reinforced arch for which the centering must be figured is only 250 lb. per sq. ft., as against 600 lb. in the Cleveland bridge, and we can expect that three-hinged steel arches for the centering of the reinforced concrete arch cost one-half of that of the Cleveland bridge.

It is possible to go still further, and instead of round bars for reinforcing the arch ribs, a latticed arch rib without hinges, as shown by Fig. 4, may be used. This may be designed to carry the dead load of the concrete ribs, and to take up the tensile and compressive stresses previously mentioned.

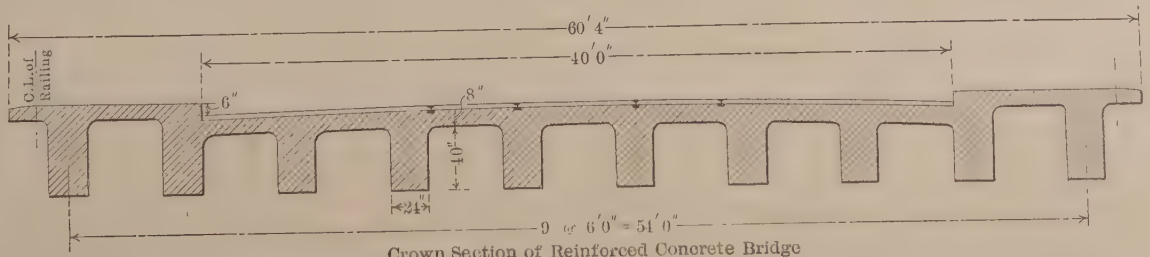
The writer has found that the maximum bending moment in the steel ribs from a symmetrical application of the dead load $= 0.0045\ pl^2$, and occurs at the crown, when 0.37 of the span is loaded at each end of the arch.* In this case the thrust $= 0.0676\ \frac{pl^2}{f}$, when f = rise in feet.

It will be advisable to concrete two arch ribs on one day and the slabs on the following working day. Let $p = 1\,000$ lb., and the maximum moment $= 0.0045 \times 1\,000 \times 280^2 = 353\,000$ ft-lb., and the corresponding thrust $= 0.0676\ \frac{1\,000 \times 280^2}{80} = 66\,300$ lb. Assuming that the theoretical depth of the latticed arch ribs $= 3.33$ ft. and the sectional area of one chord $= 14$ sq. in., it will be found that the extreme stresses from the bending moment $= \frac{353\,000}{14 \times 3.33} = 7\,560$ lb. per sq. in., and from the thrust $= \frac{66\,300}{28} = 2\,370$ lb., or the maximum compressive stress $= 7\,560 + 2\,370 = 9\,930$ lb. per sq. in., which stress, however, reduces to 4 370 lb. per sq. in. where the entire rib is concreted, and thereby the bending moment is annulled and only direct compression is acting.

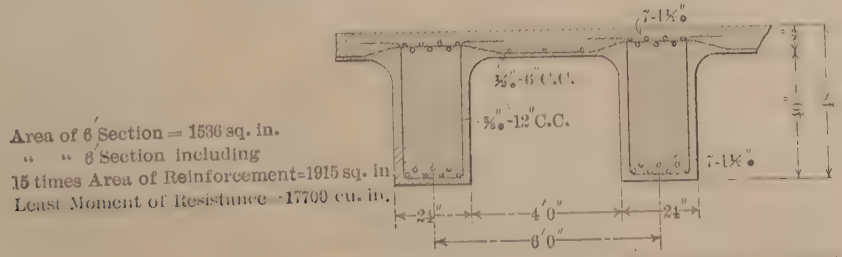
The uniform compressive stress will increase to less than 8 000 lb. per sq. in., when the entire slab is concreted on the following day. It has been shown that the greatest stresses in the round steel reinforcement were 7 000 lb. and 1 400 lb. per sq. in. in compression and tension, respectively; adopting the same values for the latticed steel arch, it will be found that the maximum compressive stress equals 15 000 lb. per sq. in., and that no tensile stresses exist in the latticed arch. No claim

* "The Reinforced Concrete Pocket Book," pp. 157-158.

Mr. Mensch.

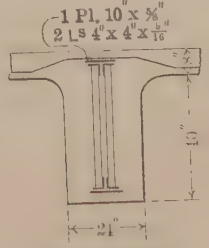


Crown Section of Reinforced Concrete Bridge of 280′ Span and 80′ Rise.
FIG. 2.



Area of 6′ Section = 1536 sq. in.
" " 6′ Section including 15 times Area of Reinforcement=1915 sq. in.
Least Moment of Resistance = 17700 cu. in.

Section of 6 Ft. Width of Bridge, Showing Reinforcement.
FIG. 3.



Section of Latticed Arch Rib without Hinges.
FIG. 4.

Mr. Mensch. is made that these stresses are exact, but it can be positively asserted that they are actually very much lower. The writer estimates the weight of nine arch ribs, as outlined, including braces, at 270 tons, and the cost at $22 000, which is practically the cost of the steel arches and wedges of the Cleveland bridge. That the reinforced concrete bridge proper was much cheaper than the solid concrete bridge is apparent from the fact that the yardage of the concrete is only one-third that of a solid concrete bridge.

Mr. Watson. WILBUR J. WATSON, M. AM. SOC. C. E. (by letter).—The writer is indebted to Mr. Mensch for his reference to the previous use of steel centers designed on principles similar to those used at Rocky River, of which he was entirely unaware.

In regard to the cost of timber centers, as compared with steel, the writer's design required the use of considerably more timber than Mr. Mensch includes in his estimate.

In regard to the shrinkage of the concrete in the keys, it was expected that there would be some shrinkage, and that, when the entire load was thrown on the struts, they would be compressed sufficiently to take up this shrinkage, owing to the very high stress to which they would be subjected. As a matter of fact, however, the shrinkage in these keys was scarcely perceptible, doubtless owing to the fact that the concrete was kept as wet as practicable.

Mr. Mensch's criticism on the writer's remark in regard to the economy of reinforced concrete *versus* plain concrete for an arch of this type is pertinent. The writer should perhaps have qualified his statement, as it was intended only that it should apply to arches designed to accommodate the heavy loads which are used by American engineers, and also in accordance with what may be styled, "established practice" among American engineers in the design of such structures.

There can be no question that the arch ring of the Rocky River Bridge might have been considerably reduced in section, had it been reinforced more heavily; and, had it been so reinforced, the designer could, of course, have departed considerably from the equilibrium polygon of the dead load. The writer, however, must confess to a preference for the massive construction, as used at Rocky River and at the Walnut Lane Bridge in Philadelphia, where natural rock foundations are to be had. This preference is due partly to asthetic considerations and partly to the general desirability of using concrete in compression as much as possible and in tension as little as possible.

What the writer desired to call particular attention to was the fact that, using a massive design, such as that at Rocky River, the live load is so small, in comparison with the dead load, that, if the neutral axis of the arch ring be made to coincide with the equilibrium polygon of the dead load, the bending moments are negligible, and, therefore, no reinforcing is required.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1204

THE PRESENT-DAY PUMPING ENGINE FOR WATER-WORKS.*

By Charles Arthur Hague, M. Am. Soc. C. E.

With Discussion by Messrs. John H. Gregory, Charles B. Buerger, George A. Orrok, and Kenneth Allen.

In the development of the modern water-works pumping engine, the refinement of tools, reduction in cost of manufacture, better and more appropriate designing, more efficient and economical shop management, have gradually led to higher and higher steam economy with lessened first cost. The commercial element has interfered more or less with the professional; nevertheless the general results show a marked improvement during the past twenty years. With the larger plants, more of a specialty in designing to suit the conditions would have produced better results. In the older and wealthier communities there is a growing conviction that low first cost is not the paramount consideration, while low cost of maintenance and high economy of fuel are. The commercial pumping engine for water-works purposes has been brought to a high level, and has been divided into four distinct types, each to suit conditions imposed, including pressure, quantity, location, first cost, fuel, maintenance, etc. These types, and rather close approximations of cost under average conditions, are as follows:

* Presented at the meeting of May 17th, 1911.

Cost of Pumping Engines Complete, with Foundations, Piping and Appurtenances, per Million Gallons per 24 Hours Capacity.

(1) Compound-condensing, low-duty engines, horizontal	$2 300
(2) Low-duty triple, condensing, horizontal	2 800
(3) Cross-compound, condensing, horizontal	3 300
(4) High-duty triple, condensing, vertical	4 800

The first and second are non-rotative or "direct acting" machinery, and the third and fourth are of the crank-and-fly-wheel type. The figures do not include anything for buildings, land, chimneys, wells, boilers, etc.

The cost of boilers with mechanical stokers, feed-pumps and appurtenances, steam piping, and minor details—everything ready for service under average conditions—would be covered by $20 per boiler-horse-power.

It is impossible to include all plants, therefore these averages are based on:

Total water load against the plungers, 90 lb. per sq. in., or a head of 207 ft., including suction and friction;

Actual evaporation in the boilers under working conditions, 8 lb. of water per pound of coal, with feed at 150° and with coal at $3 per net ton of 2 000 lb.

Steam pressure at throttle valve of engine, 75 lb. gauge, for low-duty compound; 125 lb. gauge, for low-duty triple and cross-compound; 150 lb. gauge, for high-duty triple; an allowance of 5 lb. above the pressures given for boiler pressures.

The desire is often expressed for a schedule, rate of cost, or price of pumping engines, but it is a very difficult matter to make a price list at any certain time, which will be reliable beyond an approximate guide for estimate. Although the water-works pumping engine has been brought largely to a commercial basis in manufacture and sale, the conditions under which it must operate are special for the location where wanted, and all prices pertaining to specially defined contracts are more or less changeable.

About January, 1899, prices of all sorts of material began to rise, and by 1901 were at a higher point than for several years. In the spring of 1904 a downward tendency developed, and coke ovens in Western Pennsylvania were shut down. Pig iron began to decline, Portland cement fell off decidedly in price, steel products were lower, and cast-iron water pipe was at $23 per ton; but, in the autumn of the same year (1904), the drooping markets again strengthened, and advanced steadily until the summer of 1907, when cast-iron pipe was up to $34 per ton, steel products were high and hard to get, and orders were booked for a year ahead.

In 1900, at an attempted letting, the City of Cleveland rejected all bids for pumping machinery because the prices asked were prohibitive. The needed pumping engines were bought two years later at much better figures for the city. Based on the low figures of 1896, the record of prices for pumping machinery varied as follows:

(*A*) Bottom figures of 1896 represented by..... 100%
(*B*) Later figures..... 137%
(*C*) Still later figures..... 118%
(*D*) Figures in the summer of 1907..... 155%
(*E*) Comparatively recent figures..... 115%

Thus it will be observed that the best attempt can only result in a fair average, according to the labor and material markets, and also depending on the state of the shops bidding on the special work in question. When a shop is "hungry for work," bids will be low; but when it is "full up with work," bids will be high. All shops do not strike the tide at the same stage and at the same time, hence there is a certain amount of "irregularity" in the market, with a strong or weak undertone, as the case may be.

The higher steam pressures, which go hand in hand with greater steam economy, also changed ideas on boilers, brought greater horse-power per boiler by enlarging the units, and led to restricting the dimensions of the boiler plant. In the long run, and among the many plants built, the boiler for high pressure, made up of small diameters of parts, on the unit principle, economizes space, buildings, initial cost in large powers, and other important items, to a very satisfactory degree. Under present circumstances of unit capacity, cost of construction, convenience and economy of operation, together with

considerations as to buildings and space required, the water-tube boiler, fitted with automatic stokers and damper regulators, takes the lead as a general steam generator for water-works pumping plants. The practical relation between the economic duties of pumping engines and the amount of boiler power required, may be illustrated by Table 1, the measurable amount of boiler power being indicated by taking 10 sq. ft. of heating surface for each boiler-horse-power.

TABLE 1.—Boiler-Horse-Power Required for Each Pump-Horse-Power, Counting 10 Sq. Ft. of Heating Surface per Boiler-Horse-Power.

Duty in foot-pounds per 1 000 lb. of dry steam.	Boiler-horse-power per pump-horse-power.	Duty in foot-pounds per 1 000 lb. of dry steam.	Boiler-horse-power per pump-horse-power.	Duty in foot-pounds per 1 000 lb. of dry steam.	Boiler-horse-power per pump-horse-power.
40 000 000	1.63	120 000 000	0.55	165 000 000	0.40
50 000 000	1.32	125 000 000	0.52	170 000 000	0.39
60 000 000	1.10	130 000 000	0.51	175 000 000	0.38
70 000 000	0.94	135 000 000	0.49	180 000 000	0.37
80 000 000	0.83	140 000 000	0.47	185 000 000	0.36
90 000 000	0.74	145 000 000	0.46	190 000 000	0.35
100 000 000	0.66	150 000 000	0.44	195 000 000	0.34
110 000 000	0.60	155 000 000	0.43	200 000 000	0.33
115 000 000	0.57	160 000 000	0.41		

Table 1 is based on the fact, repeatedly demonstrated, that in good ordinary practice, 1 sq. ft. of heating surface will evaporate 3 lb. of water per hour, from 150° (temperature of feed), into steam at 150 lb. gauge pressure. Therefore 10 sq. ft. will evaporate 30 lb. of water as above, and this amount of evaporation is taken as 1 boiler-horse-power.

This is a good basis, and safe in most cases; but, if at any time caution or any special reason should suggest an increase in heating surface, any desired percentage of increase may be readily added without disturbing the relations of the different rates of economy. For example: If it should be decided that 2½ lb. of water per sq. ft. of heating surface per hour is all that it would be safe to reckon on, then 20% added to the boiler-horse-power of Table 1 would provide for such a case. Or, to go to extremes, if it was thought that 2 lb. of water per sq. ft. of heating surface per hour was the limit, 50% added to the figures in Table 1 would meet the demands for boiler capacity. With boilers properly constructed and arranged, however, Table 1 will answer all reasonable purposes.

The limits of steam economy in the pumping engine are about reached, both theoretically and practically. The modern duty record is shown by Table 2.

TABLE 2.—Duty Record per 1 000 Lb. of Dry Saturated Steam.

Year.	Duty, in foot-pounds.	Year.	Duty, in foot-pounds.
1893	154 048 700	1900	178 497 000
1895	157 843 000	1900	179 419 600
1898	167 800 000	1906	181 068 605
1900	168 532 800		

The leading type is a vertical, triple-expansion, condensing engine, with outside-packed plungers, mostly of the crank-and-fly-wheel type.

In the early days of the pumping engine the tendency toward increase in size was not realized, nor what tremendous capacities the pumping units would reach; but the view is clearer now, and there is no good reason for straying too far afield in search of the appropriate line to follow. It is not easy to forecast what developments may take place; but, with the analysis of coal, a full knowledge of the heat possibilities demanded by the steam at the working pressure, the efficiency of the boilers, a clear idea of the laws of steam expansion, and with the types of the most efficient pumping engines practically fixed, it is evident that some radical departure must be made to defeat what is known at the present time.

Table 3 gives an example of how methodically the cost of plants built on the unit basis may be determined. In some cases these figures may be too high and in others too low; they are closely approximate, and enough of the data are based on records fairly to insure the figures in the table as safe for practical use in making estimates. However, the table is so close that it would be taking chances for an engineer or a contractor to guarantee the production of results for the figures named, without investigating each case by itself. The work contemplated is for the best type of modern, triple-expansion pumping engines, and high-pressure boilers. The buildings are assumed to be of good design and quality; of brick, or of stone where stone is cheap; the roofs steel-trussed and slate-covered; the chimneys adequate; and the intakes properly proportioned and thoroughly screened. The cost given includes everything except the land.

TABLE 3.—Cost of Complete Pumping Stations.

Pressure of water load pumped against, in pounds per square inch.	Cost of plant, per million gallons capacity, including reserve.	Pressure of water load pumped against, in pounds per square inch.	Cost of plant, per million gallons capacity, including reserve.
30	$6 750	90	$8 250
40	7 000	100	8 500
50	7 250	110	8 750
60	7 500	120	9 000
70	7 750	130	9 250
80	8 000		

There are cheaper classes of pumping engines, but they are necessarily of lower economic efficiency, and therefore require more boiler capacity, more coal storage, and other incidentals which, when balanced up, will tend to keep the figures about the same. A cheaper and less durable building may be used, but in the long run this will need more repairs, which when capitalized will bring the account fully up to the figures given and most likely exceed them.

It is scarcely possible that the cost of equipping pumping stations for water-works will be increased much on account of a higher type of steam machinery, because it is evident that the top limit has just about been reached, with the record a little more than 181 000 000 ft-lb. per 1 000 lb. of steam. Ten years ago it nearly touched the 180 000 000 mark; and a gain of 0.8 of 1% in ten years, with every nerve strained, is eloquent evidence of the top limit. The Mariotte curve is about the nearest approach to perfection possible for the steam engine to accomplish, in expressing the relation between the work done and the amount of steam used. If the terminal pressure is taken as expressing the steam used, and all the steam is accounted for by the diagram, then somewhere in the immediate neighborhood of 180 000 000 duty, with 96% mechanical efficiency of the machine, will be the resulting figures, with a reasonable amount of steam used in the jackets and re-heaters charged against the account.

If there were no necessity for the use of steam-jackets, or jacket steam, the figures would approach 200 000 000 rather closely, and if superheating can save jacket steam, and vitalize the working steam in the cylinders, the latter figure may be reached in the near future, as far as the official test is concerned. This pleasing result may have to be obtained, however, by the use of a surface condenser with a com-

paratively small air-pump, and this type of condenser may require more maintenance account than the jet form; and the superheat may have to be obtained at the cost of coal.

If the steam turbine can step forward at this stage of the performance, and show something better in the matter of water sent up the hill in proportion to expense incurred, it is now the proper time to do so. This would effect something of a change in the pumping station, although, as far as present evidence goes, the change will be in the direction of smaller engine-houses and larger boiler- and coal-houses. The only claim in sight for the steam turbine on pump work is a lower first cost, it being evident, thus far at least, that it cannot compete in steam economy with that form of the reciprocating engine used in pumping water for public supply. Whether the cost of construction of the steam turbine and centrifugal pump plant will be materially less than that of a reciprocating plant, when all difficulties are overcome and the turbine machine is brought to what it must eventually be, time only will tell. The writer at present is carefully investigating the turbine proposition and its possibilities as a practical water-works factor, but is not ready yet to declare results.

Unless something develops which is not yet in view, the ideal turbine pumping engine will be a steam turbine driving a centrifugal pump, all mounted on the same frame or bed-plate. There are conflicting conditions accompanying the combination of the steam and water turbine in the same machine, and it is very much the same old story learned by the reciprocating engine—the demands of a highly elastic fluid at one end of the machine and a very stubborn and practically non-elastic fluid at the other. No reciprocating steam engine can live or give good economy under the high rotative speed required for direct connection to centrifugal pumps under the ordinary heads or pressures demanded by the average water-works, so that a direct-connected reciprocating steam end, and a centrifugal water end are practically impossible. The direct-connected steam and water turbine has been used on a small scale, with doubtful economy, and is being attempted on a large scale where fuel is cheap, with what all-around success remains to be seen. It looks like the old controversy between low first cost with large fuel bills, and higher first cost with low fuel bills.

Plants equally constructed and with equally low repair and maintenance accounts can have no advantage over each other excepting in the matter of fuel economy; and the lessened quantity of coal to be bought is the real foundation on which to base an increased investment in the plant. There are only two important items which grow less by higher duty, and these are the coal account and the fixed charges on the boiler plant. Everything else increases with higher duty, excepting the wages account for equal capacities, and this remains at least as much with high as with low duty, excepting with the large high-duty triple engines, and with these the wages are somewhat less in the fire-room on account of the lessened quantity of coal to be shoveled in proportion to the pumping.

The items for and against the high-duty pumping plant are:

Against High Duty:	In Favor of High Duty:
Maintenance account for machinery.	Maintenance account for boilers.
Interest on machinery.	Interest on boilers.
Oil, waste, packing, etc.	Sinking fund for boilers.
Sinking fund for machinery.	The coal account.

To ascertain which type and class it will pay best to buy, requires comparisons and calculations which call for thought and care, but which may be readily enough made when the conditions of the contemplated plant are clearly laid down. The fixed charges against pumping machinery and boilers are as follows:

Vertical Triple-Expansion Pumping Engines:

Maintenance account	2%
Interest account	4%
Sinking fund account	3%
Oil, waste, packing, and small repairs	1%
Total fixed charges	10%

All Other Types of Pumping Engines:

Maintenance account	3%
Interest account	4%
Sinking fund account	5%
Oil, waste, packing, and small repairs	1%
Total fixed charges	13%

The reason for the difference of 2% in the sinking fund account, between vertical triple machinery and all other forms, is that it has become evident that the vertical triple type of pumping engine will probably not be replaced as obsolete by any of the other types, but the other types will eventually give way to the vertical triple. Under these circumstances, when the vertical triple is properly built, its life is taken at 33⅓ years, and that of all the other types at 20 years; representing, respectively, 3% and 5%, to make up the sinking fund to 100% in the time specified.

The fixed charges against the boiler plant are as follows:

Maintenance account	5%
Interest account	4%
Sinking fund account	5%
Total fixed charges	14%

Even when the cost of coal is $1 350 per year for each million gallons of water pumped per day, the total coal bills do not amount to very much for 2 000 000 or 3 000 000 gal. per day; but when the quantity gets toward 10 000 000 gal., the figures are more important. The larger triple-expansion pumping engines of the reciprocating, displacement type, pump 1 000 000 gal. per day with $625 worth of coal per year for the best records, and $900 is an ordinary good record; while $1 350 is about the best steam-turbine pumping record, as far as the writer can learn. It looks as though the great struggle to keep down the capital account in the turbine outfit must be partly abandoned, as far as water-works service is concerned.

If a high peripheral velocity is what the steam turbine needs, it may be necessary to increase its diameter to meet this demand, and thus reduce the rate of revolution to meet the demands of the centrifugal pump. To obtain the greatest possible effect in the steam turbine, the expansion and velocity of discharge of the steam, would require a velocity of periphery quite beyond practical limits, as centrifugal force interferes in this direction. On the other hand, the peripheral velocity of the runner of the centrifugal pump should be somewhere in the neighborhood of the velocity of a jet of water under the pumping head, represented by the well-known formula $V = \sqrt{2\,gh}$.

These are two conflicting conditions which must be met if any material economy is to be obtained by this form of pumping apparatus. Attempts to compromise are frequently made by accommodating the steam turbine work, and then reducing the diameter of the pump runner to keep down its rim speed to something like the equivalent velocity of falling bodies from the height represented by the water head pumped against. This practice results in a direct loss of mechanical efficiency in the smaller pump as against a larger pump at a lower rate of rotative speed.

Within its limitations, the centrifugal pump is an ideal machine for lifting water; which means that it must be designed and constructed for known positive conditions, if the best results are looked for. At constant speed, for a considerable range of capacity, as is often found in water-works, or for a varying head—very frequently found in the Middle West, where domestic and fire service are combined in the same apparatus—it is inefficient and unreliable. Even on stand-pipe work, as at Schenectady, N. Y., the centrifugal pump proved to be a total failure, and that city is now seriously contemplating the construction of a reservoir for the relief of these pumps; and there are others.

The centrifugal pump sends the water by centrifugal force, from the center toward the outside, and the main principles to be followed in designing are the prevention of:

(*a*) Sudden changes in water velocity,
(*b*) Sharp changes in direction of flow,
(*c*) Unnecessary friction losses.

With a steep vane there is required a less number of revolutions per minute; with a flat vane there is required a higher number of revolutions per minute, to do the same work. The above-mentioned formula ($V = \sqrt{2\ gh}$) gives an average result in feet per second of the peripheral speed of the runner; the actual results depend on the angle of the vanes and the width of the impeller.

A case in practice, which has just come to hand—the figures being taken from the annual report of the operation of a water-works plant—will be to the point, in comparisons between displacement and centrifugal pumps in practical water-works service. The data are as follows:

Pumpage per day	9 000 000 U. S. gal.
Head against displacement pumps	207 ft.
Head against centrifugal pumps	23 ft.
Cost of centrifugal pumps	$10 000.00
Cost of displacement pumps	50 000.00
Cost of fuel used for both plants	1.25 per net ton.

Cost for Fuel to Pump 1 000 000 Gal. 207 Ft. High.

With displacement pumps	$3.27
With centrifugal pumps	6.48
Difference in favor of displacement pumps	$3.21

The difference in cost of fuel per annum at 9 000 000 gal. daily pumpage, against 207 ft. head, in favor of displacement pumps, amounts to $10 544, which means that the economic duty of the displacement machinery is 1.96 times that of the centrifugal, or the ratio between a cost per million gallons 207 ft. high, of $6.48 and $3.27, as shown by the coal account given. It may be noted in passing that the mechanical efficiency of the displacement machinery is 93%, and that of the centrifugal machinery is 65%, which corresponds with the results found in general practice along these lines.

The gas engine as a water-works pumping power is entirely undeveloped on anything like the scale which it will have to reach to be seriously considered in fairly large schemes of pumpage. There will be found to be a great variation in gas production from different kinds of coal, and there is likely to be as much variation as in the generation of steam from different fuels. As a practical fact, the power developed by gas engine cylinders will be found to lessen at considerable elevations above sea level, in a certain proportion to diameter of cylinder, on account of the diminished atmospheric pressure and consequent diminished quantity of oxygen per unit of air. For example: Gas engines of 100 b.h.p. are scaled down to 80 h.p., with a fixed size of cylinder, under guaranties made for Denver, as against the operation of the same engine at or near sea level.

Developments along the line of gas-power pumping for water-works are going on, and good results in economical and reliable gas production, easy of manipulation, will be one of the improvements in the future not very far away. The large gas engine available for

water-works pumping has not yet come to the front, and there are certain adaptabilities necessary to be made—in regard to speed, for example—which complicate the problem more or less. There are now many small plants for pumping, where the gas engine fits in very well, but there are some incidental items aside from fuel economy which have weight in small plants, but which would not be considered in a large pumping plant. The small steam pumping plant is extravagant of fuel, while the small gas plant is practically as economical as the larger ones, and although the application of steam power to pumping is more direct than with gas, the great fuel economy of the gas-power apparatus enables it to operate at a profit at a much lower mechanical efficiency than the steam machine. For example: A small gas engine geared to a triplex power pump may result in a total mechanical efficiency of 75%; then, if the engine and producer give out a brake-horse-power for 1½ lb. of coal per hour, the coal per pump-horse-power will be 2 lb. The small steam pumping plant with a mechanical efficiency of 94% requires not less than 4 lb. of coal per pump-horse-power-hour, and it is doing pretty well to accomplish this.

The indicated power with steam and the brake power with gas may be taken on equal terms in the larger plants, but the direct application of power with steam and the indirect application with gas, as expressed by a mechanical efficiency of 95% for steam and 75% for gas, shows why the gas cannot compete in the larger units in the present-day pumping plant until some more direct application of its power to pumping can be made. In the smaller and more wasteful steam plants, although the mechanical efficiency holds up very high, the steam economy is low, for the reason that the smaller quantity of water will not justify the cost and complication which go with extreme high duty, but which pay on a larger scale. Under these circumstances, the gas plant shows to advantage because its coal economy is just about as good as in the larger units under practically a uniform mechanical efficiency.

The mere recording of various statements of economy in coal per power unit does not tell the entire story, as will be shown by the analysis of data. For example: Confining the comparison for the moment to steam pumping plants, the writer has two records; one of 1.02 lb. of coal per indicated-horse-power-hour, and the other 1.98 lb. Both records were obtained as nearly as could be under similar conditions in actual water-works pumping. The fuel calling for 1.98 lb. is

slack coal at $1.50 per net ton; and the fuel calling for 1.02 lb. is anthracite coal at $4.50 per net ton. The analysis of the two fuels for heat units on complete combustion is as follows:

Heat units developed by the slack..... 11 000 per lb.
Heat units developed by the coal..... 14 000 " "

In the case of the slack coal, the buyer obtained 146 000 heat units for 1 cent. In the case of the anthracite coal the buyer obtained 62 000 heat units for 1 cent. The plant consuming the slack coal used 363 heat units per horse-power-minute, while the plant with the anthracite coal consumed 238 heat units per horse-power-minute. The efficiency of the boilers was 70% with the slack and 80% with the anthracite. This, reduced to work demonstrated by indicated horse power, gives 13 200 000 ft-lb. for 1 cent for the anthracite and 9 805 680 ft-lb. for 1 cent with the slack, with the efficiencies of the boilers equalized; and this shows that a difference of 3 394 320 ft-lb. for 1 cent existed in the difference in steam economy of the two engines. The engine with the slack fuel gave 130 000 000 ft-lb. per million B.t.u., and that with the coal gave 163 000 000 ft-lb. per million B.t.u., further demonstrating the difference in steam economy in the two machines. The gain in foot-pounds for 1 cent with the engine using anthracite coal over the other is 34%, and the gain on the heat-unit basis in favor of the anthracite coal engine is 26%, which indicates good judgment in putting enough investment into the engine, boilers, and fuel to obtain good results. The engine with the coal fuel cost $4 166 per million gallons capacity per 24 hours; and the engine with the slack fuel cost $4 083 per million gallons capacity per 24 hours. This price per million gallons does not include foundations and appurtenances, so that the $4 800 per million gallons for the vertical triple pumping engine of large size given earlier in this paper is fairly well checked, as the latter figure includes everything in the engine-house belonging to the machinery. It may be of some interest to know that the two pumping engines which have just been compared were by different builders.

The present high type of water-works pumping engine apparently cannot be improved to any material extent in principle, so that the very best that can be expected is an extremely small increase in economy secured by improved construction. It is very probable that

an increase in fuel economy from 6% to 12% can be gained in many steam plants by the use of superheated steam. Some think this too low a figure for the gain by superheat, but it must not be forgotten that many of the good records for superheat have been made where engines and boilers of only fairly good economy are in use, whereas, when the best type of pumping engine is supplied with superheated steam, the application is then made to steam machinery of the very highest class, where all the steam refinements can be operated to the best advantage, and where multiple expansion, steam-jackets, re-heating, etc., are used under the very best conditions to reduce internal losses and condensation to an extremely low point. A pumping engine at the Boston high-service station at Chestnut Hill Reservoir has a capacity of 30 000 000 U. S. gal. per 24 hours, and has the following data to its credit:

Duty per 1 000 lb. of dry saturated steam (no superheat)	178 497 000 ft-lb.
Duty per 1 000 000 B.t.u.	163 925 000 " "
Duty per 100 lb. of coal	173 869 000 " "
Dry saturated steam per indicated horse power per hour	10.34 lb.
Coal per indicated horse power per hour.	1.06 "
British thermal units per indicated horse power per minute	188

If the duty for this engine, with dry saturated steam, should be increased 12% by superheating, it will read 199 916 640 ft-lb., which is about 2 000 000 above the highest record with the use of superheated steam. The highest known duty per 1 000 lb. of dry saturated steam —in round numbers 181 000 000 ft-lb.—would go up to 202 720 000 ft-lb. with 12% increase, which is considerably above the superheat results thus far obtained.

In the present-day pumping engine, the steam-jacket system has been modified somewhat from the earlier practice, and brought to a high state of efficiency by keeping the heat in the jackets close to the balancing point, where the greatest good will be done with the least loss from useless internal radiation. The distribution of the steam is now very satisfactory, and the waste room or clearance at the

cylinder ends is brought down to surprisingly low terms by the better arrangement of valve gear and steam ports.

If there is any waste heat in the smoke-flues or uptake of the boilers, re-heaters for the receiver steam can be provided, and this steam can be made a vehicle for the transportation of heat (now getting away up the chimney) back to the engine, and there made to do work. This is no special credit to the engine, beyond its providing facilities for the use of such heat as the boilers are allowing to escape; but, in the practical operation of the plant, it will reduce the coal account by turning into useful work some of the heat of combustion not absorbed by the boiler heating surfaces. It is not entirely clear why more of this practice of flue re-heating has not been done; it is certainly an old enough idea, long known. In the Pawtucket pumping engine, a small cross-compound, built by George H. Corliss in 1878, flue re-heating was successfully used. This engine gave a duty of 133 000 000 ft-lb. per 100 lb. of coal.

The quadruple-expansion pumping engine, with its additional steam cylinder, resulting either in a tandem arrangement, or an abandonment of the three-plunger design so favorable to uniform hydraulic effects, has no great future before it, and will probably never come into water-works service. With the 33 to 40 steam expansions practicable with triple machinery possessing such admirable mechanical lines, and with the conversion of heat into work about up to the practical limits, there seems to be no room for the quadruple-expansion engine. The only prominent attempt in this direction thus far made found its way to the scrap heap after a short lease of life.

Some, who are not familiar with pumping engines at work, rebel at the idea of triple-expansion engines with aims at economy; but there are other items, aside from the particular method of handling the steam, and among them the fact that the distribution of water into a force main from three plungers has never been surpassed, and probably has never been equalled. In addition to that fact, the triple-expansion pumping engine holds a record for economy of steam far ahead of compound machinery. In nearly all cases a compound would have to be fitted with two double-acting plungers on account of pulsations in the mains; horizontal engines would be rather limited in capacity, and a cross-compound, vertical machine would be objectionable to a very serious extent.

In 1878 the direct-acting, non-rotative, compound, condensing pumping engine held the front line, and successfully disputed the field with the several odd and unrepeated forms of the crank-and-fly-wheel machine which at that time aimed at high steam economy. For years the direct-acting engine, as an all-around better proposition, held sway, and its durability and reliability as a machine for pumping water for public supply have never been excelled. Gradually, however, the financial end of the crank-and-fly-wheel manufacturing establishment and the engineering end drew closer. Then the direct-acting engine took on a high-duty attachment to increase its steam economy as it became more and more hard pressed in the fight of low duty against high duty. The problem in the crank-and-fly-wheel camp was to reduce the capital account by simplifying the design, and keep on raising the duty.

The growth and drift of the public pumping engine seems to be toward larger and larger units, and the introduction of the higher duty idea into smaller and smaller units, with the vertical, triple-expansion engine rapidly taking the lead, and the cross-compound crank engine gradually forcing the direct-acting machine out of the field in the moderate and small sizes. The direct-acting, non-rotative, triple, with six steam cylinders and two double-acting plungers, stubbornly holds on, and probably will for a while longer. Just what the next ten years will bring out is not known. It looks doubtful for the dominance of the turbine type in that time if at all, and the gas engine has not yet really made a good start for the front line. Superheated steam and generally higher steam pressures up to about 175 lb. gauge pressure will take place gradually, and that will be the last firing line of the reciprocating, triple-expansion, displacement, pumping engine. It will hold this line stubbornly, and it will require a great deal more progress than is evident in any direction at present to dislodge it or even to shake it materially. Its capital and fuel accounts, even with coal at a moderate price, make a satisfactory showing, and its maintenance in the presence of good design and workmanship will not exceed 2% in large or moderate plants where it is properly cared for.

The perfect present-day pumping plant involves the following items:

Vertical, triple-expansion, crank-and-fly-wheel pumping engines;

Long stroke, with rotative speed not to exceed 20 rev. per min.;
Maximum piston travel 200 ft. per min.;
Modified steam-jacketing and re-heating;
Steam pressure at throttle, 175 lb. per gauge;
Moderately superheated steam by independent apparatus;
Smoke-flue re-heating;
Water-tube boilers;
Mechanical stokers;
Natural draft at least 0.8 in. of water;
Feed-water economizers;
Automatic damper regulators;
Coal bought on the basis of 14 000 heat units per lb.;
Boiler efficiency of 75%;
Coal per indicated horse power, 1 lb. for large plants;
Coal per indicated horse power, 1.75 lb. for small plants;
Maintenance of engines, 1.5% for large plants;
Maintenance of engines, 3% for small plants.

These and other items of a similar nature are about what a look ahead discerns as the coming events in the planning, construction, and operation of present-day pumping plants for public water supply.

Chicago, St. Louis, Cincinnati, New Orleans, Cleveland, Buffalo, Pittsburg, Philadelphia, and other cities, large and small, will have to depend on pumping for the main supply; and New York, Boston, Baltimore, and other cities, in which the growth extends to levels too high for distribution from the general source of supply, will have to use auxiliary service involving pumpage for high-service supply. Therefore, it will be noted that the use of the best present ideas, and the introduction of new ones, will be in order for a long time to come, in the municipal water-supply pumping station.

DISCUSSION

Mr. Gregory.

JOHN H. GREGORY, M. AM. SOC. C. E.—The author has presented an admirable paper on a subject which is of great interest, not only to water-works engineers who are engaged in operating works, but also to those who are called on to design them. It is to be regretted that he has confined his text to the consideration of pumping engines which serve either to deliver water directly into distribution systems or into distributing reservoirs, and has not discussed the low-lift pump or pumping engine. The demand for water of a higher standard of purity has necessitated, and will still necessitate, the installation of water purification works, and in the majority of such works it is probably safe to say that low-lift pumps will be required; therefore a discussion of the low-lift pump or pumping engine, especially of the centrifugal pump operated by steam, electric, or water power, would be welcome.

The author has given some good general figures as to the cost of various types of pumping engines complete, with foundations, piping, and appurtenances. That high-duty, vertical, triple-expansion, condensing pumping engines, under the conditions stated by Mr. Hague, have been installed at a cost of $4 800 per million gallons per 24 hours, the speaker does not question, but he is inclined to believe that, for a general average, the figure stated is a little low. Table 3 gives the cost of pumping stations complete, including reserve, equipped with pumping engines of this type. The value of this table would be materially increased by the addition of another column giving the percentages of reserve allowed for. The amount of reserve will vary greatly, depending on the capacity of the pumping station. For example, with a net capacity of 20 000 000 gal. per 24 hours, two 20 000 000-gal. pumping engines might be installed, in which case the reserve would be 100 per cent. With a much larger station, having a net capacity of, say, 100 000 000 gal. per 24 hours, a reasonable installation would include six 20 000 000-gal. pumping engines, five in service and one in reserve; in which case the reserve would be only 20 per cent.

The author's comments on the steam turbine and centrifugal pump are interesting. That the high-duty, vertical, triple-expansion, pumping engine, with its high first cost and great efficiency, is here to stay, is hardly to be questioned; but the speaker believes, also, that there is a field for the less costly and less efficient turbine-operated centrifugal pump. It is probably safe to say that, at present, in most cases, preference would be given to the vertical, triple-expansion, pumping engine, but it is not hard to conceive of conditions under which it might be more desirable to install the turbine-operated centrifugal pump, especially where the cost of coal is very low. As bearing on the cost of such a pump, the following figures, given to the speaker by the

engineer who installed the work, may be of interest: The unit consists of a single-stage, centrifugal pump and a 2 500-h.p., steam turbine (operating under superheated steam), with a speed of 15 000 rev. per min.; and has a capacity of 18 000 000 gal. per 24 hours against a head of 405 ft. The unit cost was $50 000, and the duty developed was 110 000 000 ft-lb. per 1 000 lb. of steam. The speaker understands that the unit has been in service about two years, and that its operation has been satisfactory. As to whether this installation will prove to be a good paying investment in the long run, time alone will tell. Mr. Gregory.

Charles B. Buerger, Assoc. M. Am. Soc. C. E. (by letter).—The following remarks may be of interest in amplification of Mr. Hague's very able résumé of pumping engines. Mr. Buerger.

The duty record, as given in Table 2, stopping at 1906, will, if brought up to date, show some improvement achieved since that time:

1906: Allis Chalmers, triple-expansion, vertical engine, for the Hackensack Water Co., at the Milford, N. J., station.—Rated capacity = 12 000 000 gal. per day; stroke = 48 in.; speed = 25.45 rev. per min.; steam pressure = 176.67 lb., gauge; water pressure = 334.75 ft.; cylinder diameters = 30, 54, and 84 in.; plunger diameter = 26 in.; duty per 1 000 lb. of dry steam = 181 433 826 ft-lb.

1909: Holly, triple-expansion, vertical engine, for the City of Albany, N. Y.—Rated capacity = 12 000 000 gal. per day; stroke = 60 in.; cylinder diameters = 28, 54, and 80 in.; plunger diameter = 25 in.; speed = 22.14 rev. per min.; steam pressure = 153.56 lb., gauge; exhaust pressure = 3.24 in. of mercury, absolute; water pressure = 322.21 ft.; duty per 1 000 lb. of dry steam = 182 281 000 ft-lb.

1910: Holly, triple-expansion, vertical engine, for the City of Philadelphia, Pa.—Rated capacity = 20 000 000 gal. per day; stroke = 66 in.; cylinder diameters = 30, 60, and 90 in.; plunger diameter = 33 in.; speed = 20.13 rev. per min.; steam pressure = 180.19 lb., gauge; exhaust pressure = 2.52 in. of mercury, absolute; water pressure = 220.85 ft.; duty per 1 000 lb. of dry steam = 184 476 000 ft-lb.

Mr. Hague hardly does justice to the possibilities of the turbine-driven centrifugal pump. It is true, as he says, that it has not established itself as a competitor on equal terms with the high-grade reciprocating engine for all conditions of service, but it has earned a sufficient place to require its serious consideration for any particular installation, and in many cases it will be found that the turbine pump unit will show the best ultimate economy, when combining with the operating charges the fixed charges on the investment.

A comparison of the two types of installation under the same conditions of service will illustrate this statement. It is assumed in this comparison that the service is such that the pumps can operate at their full rated capacity when working, and against their rated pressure.

Mr. Buerger.

Capacity of pump = 10 000 000 gal. per day; water pressure = 207 ft.; steam pressure = 150 lb., gauge; vacuum, supplied with pump.

The pumping engine is assumed to be in operation 80% of the total time during the year. It is probable that, in the average installation of pumping stations, the delivered capacity is considerably less than 80% of the rated capacity of the station, and this assumption is somewhat in favor of the pump with the higher steam economy and higher first cost.

(*a*) Triple-expansion, vertical, single-acting engine; 48-in. stroke; 25 rev. per min.

Cost, installed, including foundations and appurtenances, from Mr. Hague's Table 2 = $50 000; engine floor space = 18 by 40 ft.; engine height = 40 ft.; engine-room floor space = 38 by 60 ft.; building height, allowing space for crane = 53 ft.; building contents for each engine = 120 000 cu. ft.; engine-room cost = $24 000; duty guaranteed by builder = 160 000 000 ft-lb. per 1 000 lb. of dry steam; average operating duty, assumed at 20% less than guaranteed duty, or 128 000 000 ft-lb. per 1 000 lb. of dry steam. (The test duty would exceed the guaranteed duty by 5 per cent.) Dry steam to engine per day = 134 000 lb.; allowance for auxiliaries and losses (10%) = 13 400 lb.; boiler-horse-power developed = 205; average boiler-horse-power required at 80% operation = 164; assumed total cost of steam, allowing for fixed charges $50 per horse-power per year: Total steam cost per year = $8 200; boiler capacity installed = 200 h.p.; cost of boiler-room = $6 000.

Charges against engine per year: Interest = 4%; sinking fund, based on a life of 20 years and a 2% return on the sinking fund = 4%; maintenance and supplies, not including operating labor = 4%; total = 12 per cent.

Charges against buildings: Interest = 4%; sinking fund, based on a life of 30 years and 2% return = 2½%; maintenance = 1%; total = 7½ per cent.

Yearly charges against reciprocating engine plant:

Engine = 12% on $50 000	$6 000
Buildings = 7½% on $30 000	2 250
Steam charges =	8 200
Total	$16 450

(*b*) Compare with a turbine pump unit installation of the Delaval type, of the same size, and running under the same conditions.

Cost, installed with appurtenances = $20 000; floor plan = 15 by 25 ft.; height of pump = 10 ft.; building space = 27 by 37 ft.; building height = 25 ft.; engine-room contents = 25 000 cu. ft.; engine-

room cost = $5 000; duty guaranteed by builder = 100 000 000 ft-lb. per 1 000 lb. of dry steam; average operating duty, assumed at 20% less than the guaranteed duty, or 80 000 000 ft-lb. per 1 000 lb. of dry steam; dry steam per day of full operation = 215 000 lb.; corresponding boiler capacity = 298 h.p.; allowance for auxiliaries and losses = 10%; total boiler capacity developed for full operation = 328 h.p.; average boiler capacity developed on 80% basis = 262 h.p.; yearly steam charges at $50 per h.p. = $13 120; boiler capacity installed = 450 h.p.; boiler-room cost = $9 000. Mr. Buerger.

Yearly charges against turbine plant:

Pump unit = 12% on $20 000	$2 400
Building charges = 7½% on $14 000	1 050
Steam charges =	13 120
Total	$16 570

These figures seem to indicate that the more economical type of turbine centrifugal pump can sometimes meet the high-grade reciprocating pump. It is not possible to draw any general conclusions, however, from such a particular example.

Mr. Hague, in his discussion of the centrifugal pump, has fallen into an error in the statement that the attempt to meet the high speed of the steam turbine by reducing the diameter of the pump runner results in a loss of mechanical efficiency, as against a larger pump at a lower rotative speed. The correct relation is that the higher the rotative speed and the smaller the impeller diameter the greater will be the mechanical efficiency. The only limitation is that the impeller must be somewhat larger in diameter than the suction opening required.

In considering the benefit of superheat, Mr. Hague credits a fuel economy of from 6 to 12% to superheating. When used in connection with engines of the type taken by Mr. Hague in his comparison, such as the Chestnut Hill high-service engine, it is probable that 6% is the upper limit of the fuel economy that can be expected.

The most satisfactory published trials of superheated steam in efficient pumping engines are those of 1908 of the Camden Iron Works engines at the River Station in Cincinnati.

The main features of these trials are as follows: Rated capacity = 30 000 000 gal. per day; water pressure = 140 ft.; cylinder diameters = 29, 54, and 82 in.; stroke = 96 in.; plunger diameter = 37½ in.; speed = 15½ rev. per min.

No. 3 engine was tested with both saturated and superheated steam, with the following results:

	Saturated.	Superheated.
Steam pressure, gauge	151.4 lb.	148.9 lb.
Superheat at engine		95.8°
Vacuum, in pounds	13.81	14.1

Mr. Buerger.

	Saturated.	Superheated.
Duty, per 1 000 000 B.t.u., in millions of foot-pounds.....	154.6	163.
Increase in duty.....	.....	5.44%
Duty per 100 lb. of coal, in millions of foot-pounds....	156.3	166.4
Correction for boiler plant operation	.....	4.34%
Increase in engine efficiency, fuel basis.....	.....	6.15%

With engines such as the Philadelphia engine noted before, which, with saturated steam, shows a duty of 163 330 000 ft-lb. per 1 000 000 B.t.u., which is as high as this Cincinnati engine with superheat, the probable gain from superheating would be less than the amount shown by these tests.

Mr. Orrok.

George A. Orrok, M. Am. Soc. C. E.—The speaker's acquaintance with water-works pumping engines is small, but a word must be spoken for the turbine-driven centrifugal pump, which is commencing to come into vogue. The capacity of the pump the speaker has in mind was 1 000 gal. per min. (1 440 000 gal. per 24 hours) against a head of 700 ft. Its guaranteed efficiency was 65 per cent. Under service conditions, an efficiency of 68% was obtained. The pump was driven by a turbine, running non-condensing, at 1 650 rev. per min. The turbine used about 33 lb. of steam per b.h.p. The duty per 1 000 lb. of steam was about 40 000 000 ft-lb.

With careful design and good construction, the centrifugal pump will develop an efficiency as high as 75 or 80%, say, 78%, as a fair limit, with speeds for turbine drive of from 2 000 to 2 500 rev. per min. A well-designed condensing turbine, at these speeds and in the sizes from 500 to 2 000 b.h.p., should develop 1 b.h.p. for 9 lb. of steam or less, and the duty of such an outfit would be about 170 000 000 ft-lb. per 1 000 lb. of steam.

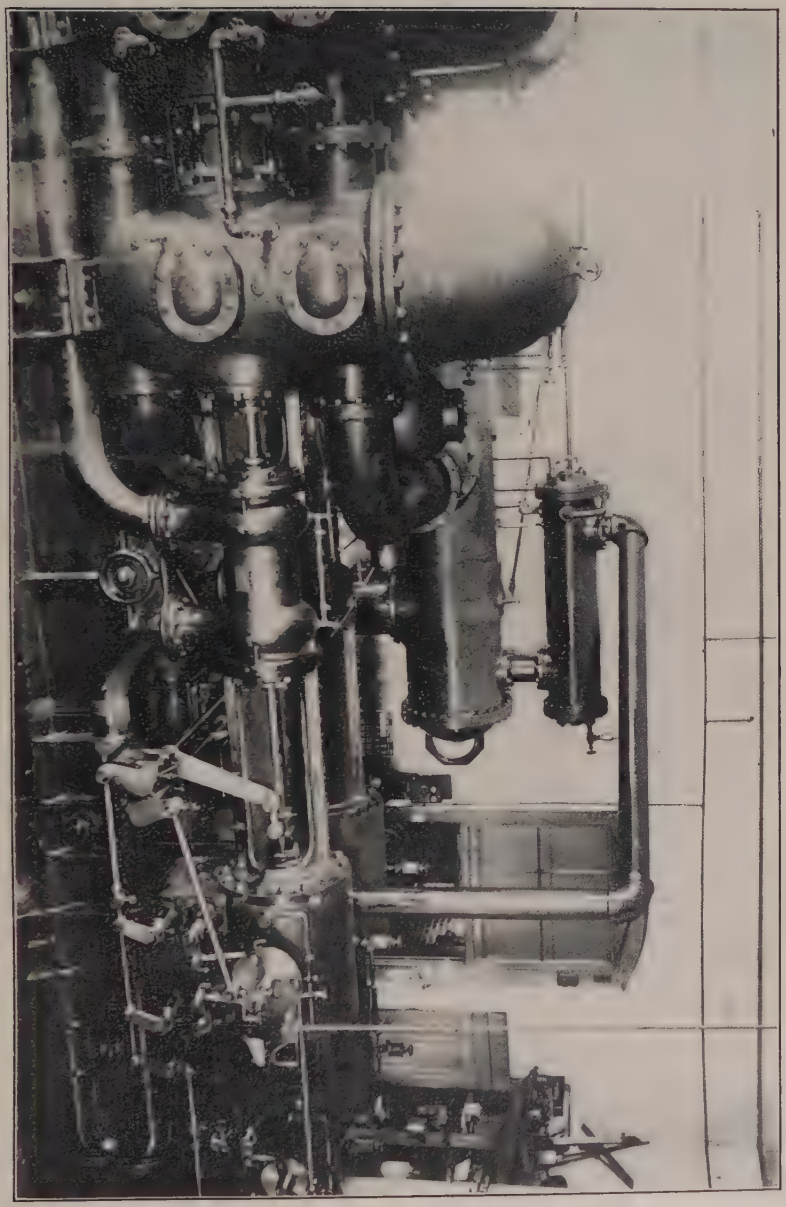
The feed-pump referred to above occupied a space of 15 by 3 by 3 ft., and had five stages for 700 ft. head. To-day such a pump would have only two stages, would be 2 ft. shorter, and 1 or 2% more efficient.

A 20 000 000-gal. pump, for heads between 300 and 400 ft., would be approximately 18 ft. long, and probably not more than 5 ft. in diameter.

The good turbine efficiency mentioned above is attained now by practically all makers.

Mr. Allen.

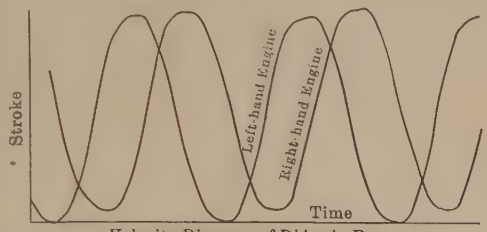
Kenneth Allen, M. Am. Soc. C. E.—Would not the combination of a gas engine operating triplex pumps through eccentrics on the main shaft be a good one for accommodating the high engine speed to the reciprocating motion of the plunger?



D'Auria Pumping Engine, With Compensating Device.

Mr. Allen.

The compensating device of the d'Auria pump is an interesting and valuable feature which probably has not received the recognition it deserves. This, as mentioned by the author, consists in an oscillating column of water actuated by a plunger inserted between the steam and water cylinders. This responds promptly, automatically, and to the degree necessary to prevent any sudden or material variation of speed due to change of load.



Velocity Diagram of D'Auria Pump
Water Pressure 46 Lb. per Sq. In.
Steam " 82 " " " "
Revolutions per Minute 72

FIG. 1.

A speed diagram (Fig. 1), taken from the motion of the piston of a d'Auria pump at Atlantic City, shows a remarkably smooth variation of velocity at all parts of the stroke, with no stop at the end, such as usually occurs with direct-acting pumping engines of this type.

This pump was built by Henry G. Morris, of Philadelphia, after designs by the inventor, Mr. Luigi d'Auria, and was the second of the kind built, the first being for the Philadelphia Water Department. It was placed in service on January 1st, 1902. On January 29th, 1903, a 10-hour test was made by the speaker in which the duty developed was 73 679 000 ft-lb. per 1 000 lb. of commercially dry steam, the capacity shown being 3 411 000 gal. per day with steam at 88.6 lb. pressure and a total lift, including suction, of 119.06 ft. The efficiency of the steam end was 85.4%; of the water end 93.6%, and of the two combined, 80.0 per cent. These results were probably fair, considering the small size of the pump and the low steam pressure, but have doubtless been exceeded in more recent plants.

Soon after this pump was put into commission a blank flange on the 20-in. force-main blew off, resulting in a sudden drop in the water pressure such as would have caused a dangerous increase in the speed and stroke of an ordinary duplex pump. In this case the adjustment to the situation was prompt, easy, and effective, with no damage to the pump.

Charles Arthur Hague, M. Am. Soc. C. E., the author of this paper, died June 25th, 1911. At the meeting of May 17th, 1911, Mr. Hague presented his paper and took part in the discussion of it, but subsequently he had no opportunity to revise his remarks, as reported by the stenographer, and therefore they cannot be given here.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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TRANSACTIONS

Paper No. 1205

TWO EARTH DAMS OF THE UNITED STATES RECLAMATION SERVICE.*

By D. C. Henny, M. Am. Soc. C. E.

With Discussion by Messrs. James Dix Schuyler, and D. C. Henny.

The object of this paper is to describe in some detail two earth dams built by the United States Reclamation Service, and to compare the different methods used in their construction. The two dams are the Cold Springs Dam, in Oregon, and the Conconully Dam, in Washington.

Cold Springs Dam.

The Cold Springs Dam is part of the works of the Umatilla Project, which comprises a tract of land lying south of the Columbia River and east of the Umatilla River, encompassing the Town of Hermiston, 192 miles due east of the City of Portland.

The principal dimensions of the dam are:

Greatest height above bottom of creek channel	98.5 ft.
Height above valley bottom	87.5 "
Width of valley on center line of dam	400.0 "
Length of crest of dam	3 800.0 "
Top width	20.0 "
Length of spillway	330.0 "

* Presented at the meeting of June 7th, 1911.

Up-stream slope	3:1
Down-stream slope	2:1
Total volume of dam	673 200 cu. yd.
Capacity of inlet canal, at reservoir	300 sec-ft.
Capacity of outlet canal	225 sec-ft.
Capacity of reservoir above outlet gates	49 000 acre-ft.
Area of reservoir at high-water level	1 590 acres.
Drainage area above dam	210 sq. miles.

Source of Irrigation Supply.—The Columbia River, bordering on the irrigable lands to the north, has its water surface from 140 to 240 ft. below the general ground surface. Its grade is very flat, and gravity diversion from this source, therefore, is entirely out of the question. The Umatilla River has a fairly steep grade, averaging 15 ft. per mile in its lower 25-mile course, and presents the only possibility for gravity diversion.

From November 1st to July 1st the flow usually varies from 50 to 5 000 sec-ft., March and April being the flood months. During the summer the flow is less than 50 sec-ft., and is fully appropriated by prior irrigators.

It was computed that with the water which might be directly diverted for irrigation during the flood season, a storage supply of 49 000 acre-ft. would be sufficient for the irrigation of the 25 000 acres of irrigable land embraced in the project, considering also a prior water right which had been purchased.

Reservoir Site.—On the main river, above the point of desired diversion, a reservoir site could not be found, but a suitable one was located in the small adjoining basin of Cold Springs Creek, above and close to the lands to be irrigated, as shown on Fig. 1. This basin has an area of 210 sq. miles, and presents a gently rolling topography.

The precipitation on the water-shed of the creek ranges from 6 in. for the lower, to 18 in. for the higher, lands, much of which is dry-farmed for grain. The land has a generally deep and retentive soil, the rock crops out at only a few points, and the slopes are so flat, especially near the divides, that a survey was required to establish the limits and area of the water-shed with some degree of certainty.

The creek is usually dry throughout the year, and all evidence indicated a very limited flow at any time. There was some underflow,

however, and small springs break out lower down, from which the creek probably derived its name. Usual absence of surface run-off is explained by the small rainfall, the gently sloping character of the

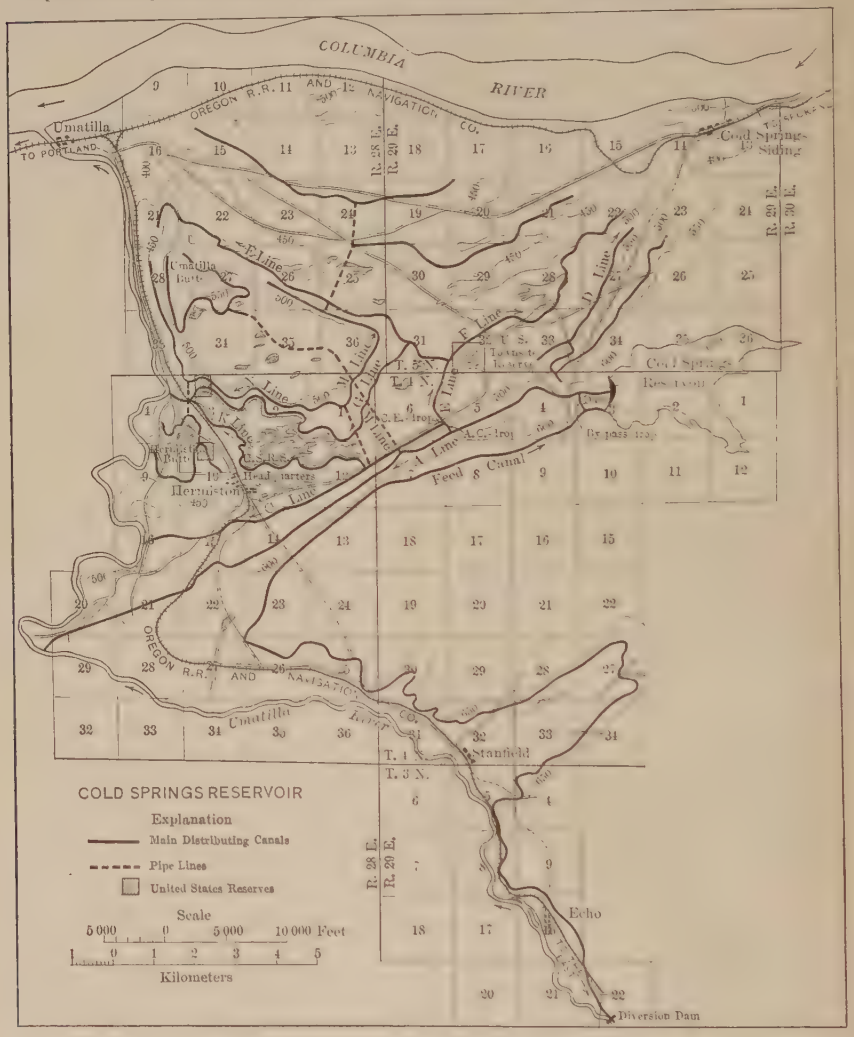

FIG. 1.

ground surface, and the absorbent and retentive character of the soil. The reservoir is supplied by a feed canal from the Umatilla River, with a capacity of 300 sec-ft. at the reservoir and a length of 25 miles, presenting no special difficulties. It heads about 1½ miles above the

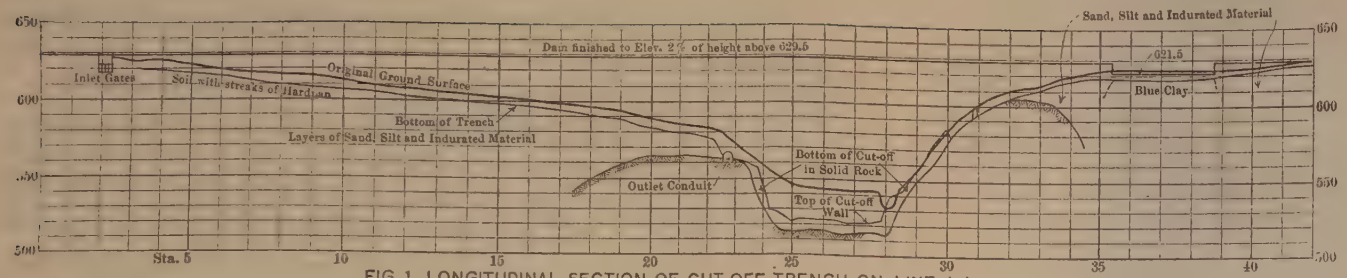

FIG. 1. LONGITUDINAL SECTION OF CUT-OFF TRENCH ON LINE A-A

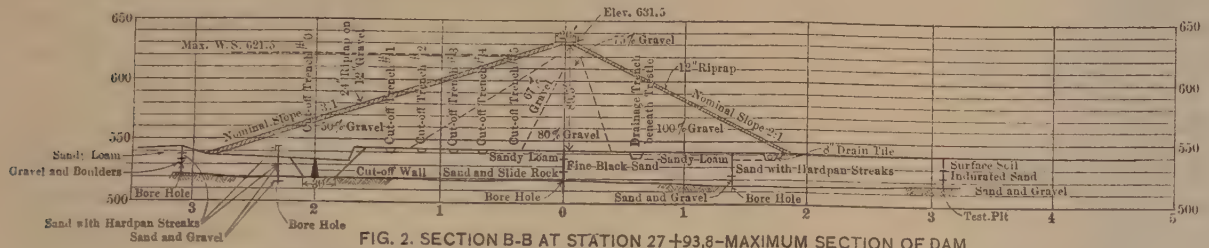

FIG. 2. SECTION B-B AT STATION 27+93.8—MAXIMUM SECTION OF DAM

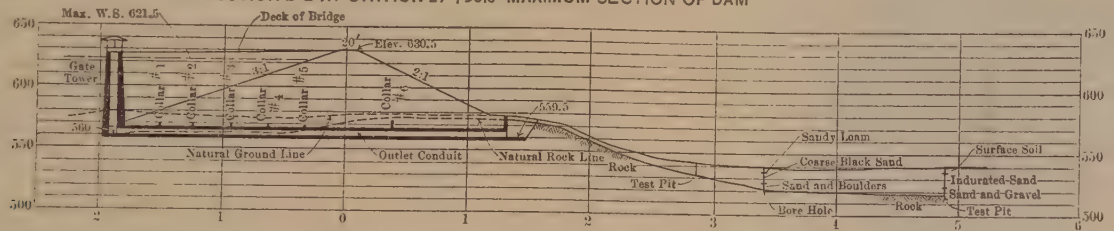

FIG. 3. SECTION C-C THROUGH OUTLET CONDUIT

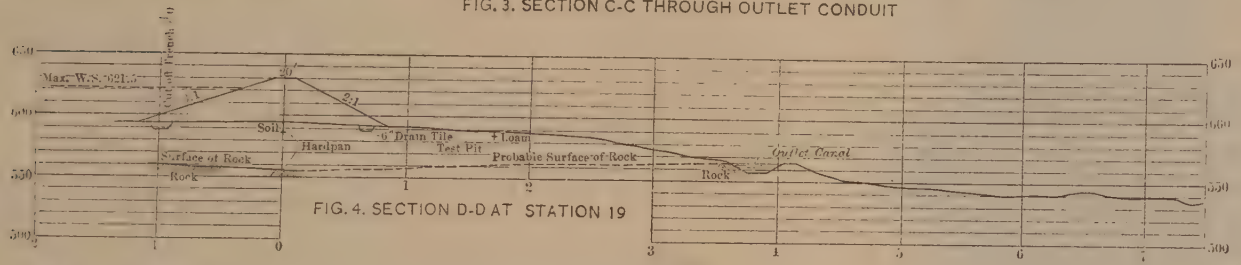

FIG. 4. SECTION D-D AT STATION 19

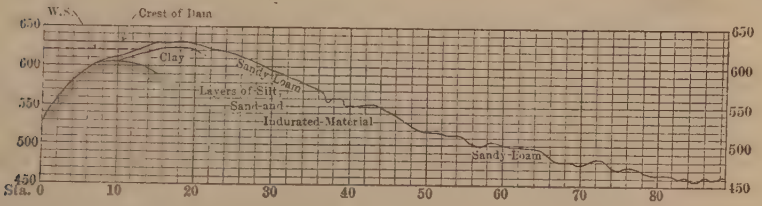

FIG. 5. SECTION THROUGH RIDGE AT NORTH END OF DAM ALONG LINE E-E

SECTIONS OF
COLD SPRINGS DAM.

Town of Echo, and discharges into the reservoir near the south end of the dam.

The dam is at the upper end of a cañon, above which the valley spreads out, the cañon continuing down stream for about 1 mile, after which the valley opens up, there being no special watercourse until the head of a second cañon is reached, through which the water finds its way into the Columbia River. At the dam site the valley is 400 ft. wide and has an elevation of 542 ft. above sea level, the bottom of the creek channel being at 531 ft. In order to produce the required storage above an outlet elevation of 560, it was found that high water would come at Elevation 621.5 ft., when the reservoir would hold 49 000 acre-ft. and have a surface area of 1 590 acres.

To the south and east the land is generally rising, but to the north a ridge forms the side of the reservoir, broad at its eastern end and narrowing up toward the dam site, the base at the valley floor elevation being about 1 mile wide at its narrowest point and 450 ft. at high-water elevation. The rock in this ridge crops out at a few points on the reservoir side, and evidently rises toward the east, but in the narrow part of the ridge it is at unknown depth. The position of the rock on the center line of the dam is shown in the longitudinal section, Fig. 1, Plate VII.

The rock is of igneous origin, non-columnar in character, hard, sound, and of good weight. On blasting, it breaks up into sharp angular fragments of assorted sizes, the largest seldom containing more than 1 cu. ft. On exposed faces the rock appears tight, the only open seams in evidence occurring on the south cañon side near the lower toe of the dam.

The overlying material, from the surface down, generally consists of fine, loamy soil, of sandy character, to a depth of from 2 to 8 ft., fine sand, irregular layers of silt from a few inches to several feet in thickness, coarser sand, and, in the bottom of the cañon, water-bearing gravel. Many of the strata were found in various degrees of induration. Their general position is horizontal, but they do not continue unbroken for any great distance. The only clay found was a small body on the north hillside, hard and blue, and resembling slate. It has been made to serve as a foundation for the overflow spillway.

The longitudinal profile of the dam shows that while it is feasible to make tight juncture with the rock crossing the cañon and on the

cañon sides, this is not the case at some distance away, where the rock drops beyond economical reach.

From the foregoing it may be noted that the site is not a very desirable one, from the standpoint of assured water-tightness. Assuming a perfectly water-tight dam to be built, connecting with bed-rock across the cañon and reaching to depths of from 15 to 6 ft. on the side-hill, it is apparent that leakage from the reservoir remains possible through the seams in the rock, through the layers of sand on each side of the cañon below the cut-off line, as shown by the profile, Fig. 4, Plate VII, and through the ridge to the north of the reservoir in the vicinity of the dam, as shown by the profile, Fig. 5, Plate VII. These possibilities of leakage were carefully studied.

As regards the rock, volcanic formations in the Northwest are generally very changeable within short distances, and the absence of seams and the soundness of the rock within the range of local borings give no assurance of its character within a short distance beyond. This condition prevails throughout large areas in Oregon, Washington, and Idaho, and forms one of the elements of risk in connection with all reservoir construction in that region.

As regards the overlying material, movement of water through some fine sand layers was to be expected, both under the dam and through the north ridge. In either case it was argued that the path to be followed by the water would be circuitous and long, as compared with the head. Thus velocities would be held down to ordinary percolating limits, such as would be utterly incapable of carrying the granular material of which the soil is mostly composed.

As regards the economical value of possible leakage water, the reservoir will be filled slowly during the winter, but the bulk of the river water is not available for reservoir supply until near the beginning of the irrigation season, when leakage under and around the ends of the dam, which would gather in the cañon below, can be diverted and used for irrigation.

Available Material for Dam Construction.– There were available for dam construction, within reasonable distance, the following classes of material:

Basaltic rock, hard and sound, readily blasted, and quite suitable for rip-rap;

PLATE VIII.
TRANS. AM. SOC. CIV. ENGRS.
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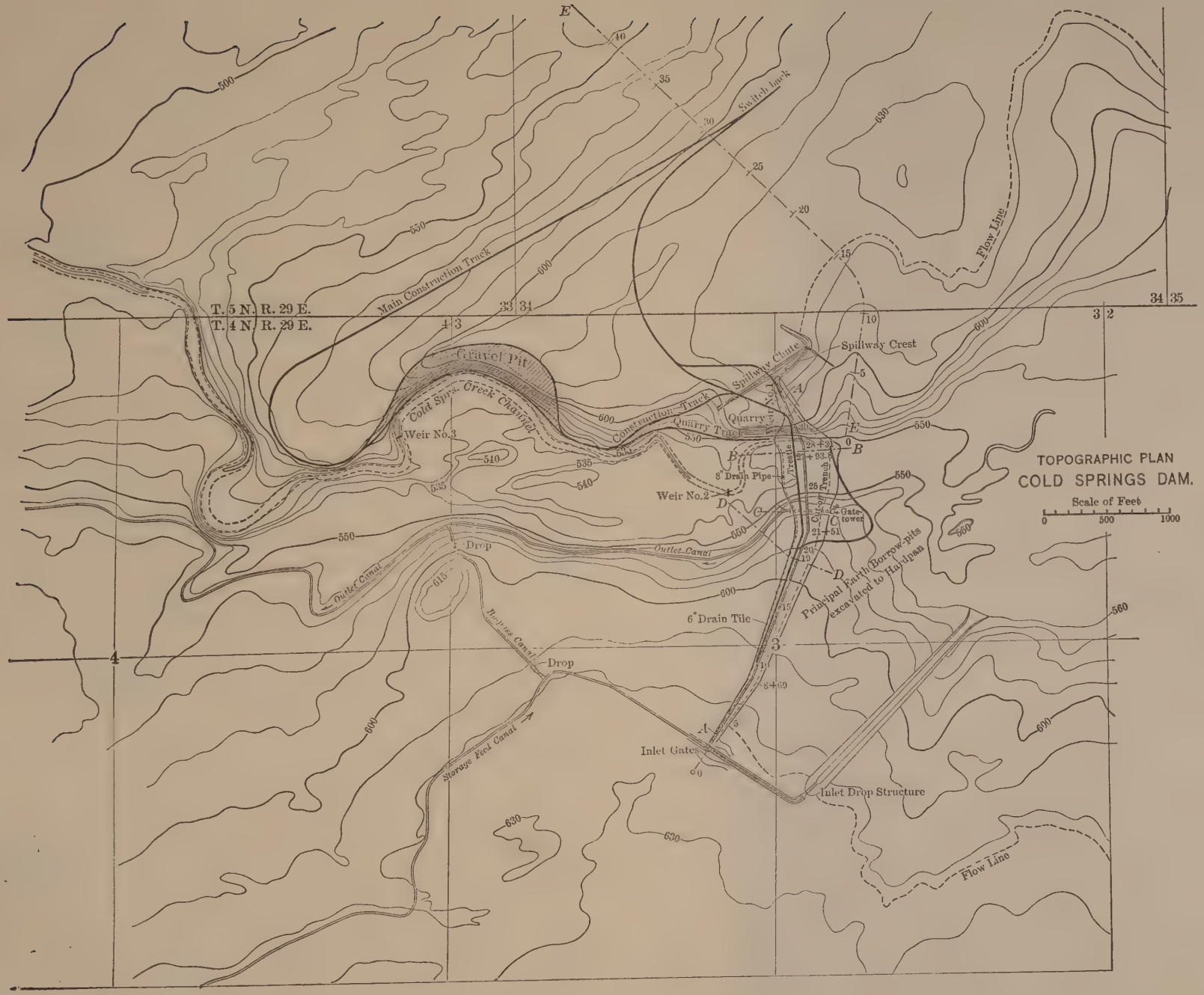
HENNY ON
TWO EARTH DAMS
OF THE U. S. RECLAMATION SERVICE.



Gravel in deep hillside deposits on the north side of the cañon, ½ mile below the dam, as shown on Plate VIII;

Fine sandy loam, most readily available in any direction, for use in the embankment;

Pure volcanic ash, in occasional strata and in small quantities;

Indurated clay deposits, principally at the north end of the dam, and limited in area;

Sand and gravel, in deeper strata underlying the surface soil, and to some extent indurated.

The first three were the only ones which could be obtained in large quantities without removing large masses of overlying materials. In order to judge of their suitableness for dam construction, a series of percolation experiments was made. These experiments have been fully described in an article* by E. G. Hopson, M. Am. Soc. C. E., and the writer.

Five classes of material were tested in an 18-in. tank similar to that used in experiments by the Massachusetts State Board of Health. In addition, tests were made with various mixtures of fine subsoil and gravel. Physical characteristics, specific gravity, and coefficients of fineness and uniformity, were also determined. The results, from the article mentioned above, are quoted in Table 1, which has been corrected. These results are also shown on Fig. 2.

TABLE 1.—Percolation Experiments, Cold Springs Dam.

Sample.		Specific Gravity.		Percentage of Voids.			Uniformity coefficient.	Effective size, in millimeters.	Rate of percolation, in gallons per acre per day.
				Dry.		Wet.			
		Constituents.	Mass.	Loose.	Compact.	Rammed.			
(*A*) Surface soil....		2.52	1.41	59	49	44	3.9	0.023	180 000
(*B*) Fine subsoil...		2.79	1.55	54	43	45	4.0	0.025	350 000
(*C*) Pit gravel.....		2.90	1.91	42	37	34	9.2	0.559	155 000 000
(*D*) Volcanic ash...		2.66	0.94	74	68	65	5.5	0.016	400 000
(*E*) Coarse subsoil.		2.93	1.57	55	49	46	1.5	0.079	650 000
Mixtures:									
B	*C*								
75%	25%	2.64	1.75	50	39	38	3.0	0.066	60 000
67%	33%	2.83	1.76	47	41	39	....	.....	30 000
50%	50%	2.90	1.91	42	33	34	14.7	0.076	20 000
33%	67%	2.87	1.95	41	35	32	20.0	0.076	30 000
25%	75%	2.88	2.01	47	43	29	21.0	0.086	30 000
20%	80%	2.91	2.04	47	40	30	....	0.091	70 000
15%	85%	2.84	1.88	41	35	34	18.0	0.102	1 000 000

* *Engineering News*, March 7th, 1907, p. 250.

In explanation, it should be stated that the specific gravity of the mass refers to the material in its most compact form in dry condition. For the determination of the percentage of voids in the dry loose material, it was poured into the tank from a scoop when perfectly dry, and for the dry compact material, it was rammed into thin layers with a stick having a 2 by 2-in. base. The wet rammed material was similarly deposited after such degree of moistening as was believed to result in the tightest mass. The uniformity coefficient and effective size were determined by the Massachusetts State Board of Health standard.

The pit gravel is made up principally of coarse sand, containing about 10% of pebbles and cobbles. It has a far smaller percentage of voids than the finer material, due to a more favorable graduation of sizes. It also shows the highest specific gravity, both as to its ingredients and as to the mass, thus fitting it well for use in a dam, to furnish stability and perfect drainage.

The rate of percolation is for a head of 3 ft. through 3 ft. of material on the third day after the commencement of the test, and is corrected for temperature on the basis of the Hazen formula.

It will be noted from the results of the experiments that the addition of subsoil to the gravel has a very marked effect in reducing the rate of percolation. Practical water-tightness was obtained by a wide range of proportions, there being little difference in the rate of percolation between a mixture containing 25% and one containing 66% of soil. Thus the exact proportion of the mixture within these ranges was not essential, which is an important consideration in the practical work of dam construction. The greatest tightness was found with a proportion of 50% gravel and 50% soil, with which the rate of percolation, as compared with that of soil alone, was found to be less than 6%, and as compared with gravel alone less than 0.03 of 1 per cent. After careful study of the results of these experiments, a section of dam was adopted, as shown on Fig. 2, Plate VII. Stability was secured by the use of gravel throughout the entire section of the dam; water-tightness by an admixture of fine subsoil in the up-stream portion of the section; and perfect drainage by the use of unmixed gravel in the down-stream portion.

The cut-off trench across the bottom of the cañon is 29 ft. deep and 30 ft. wide at its connection with bed-rock. It reduces in depth

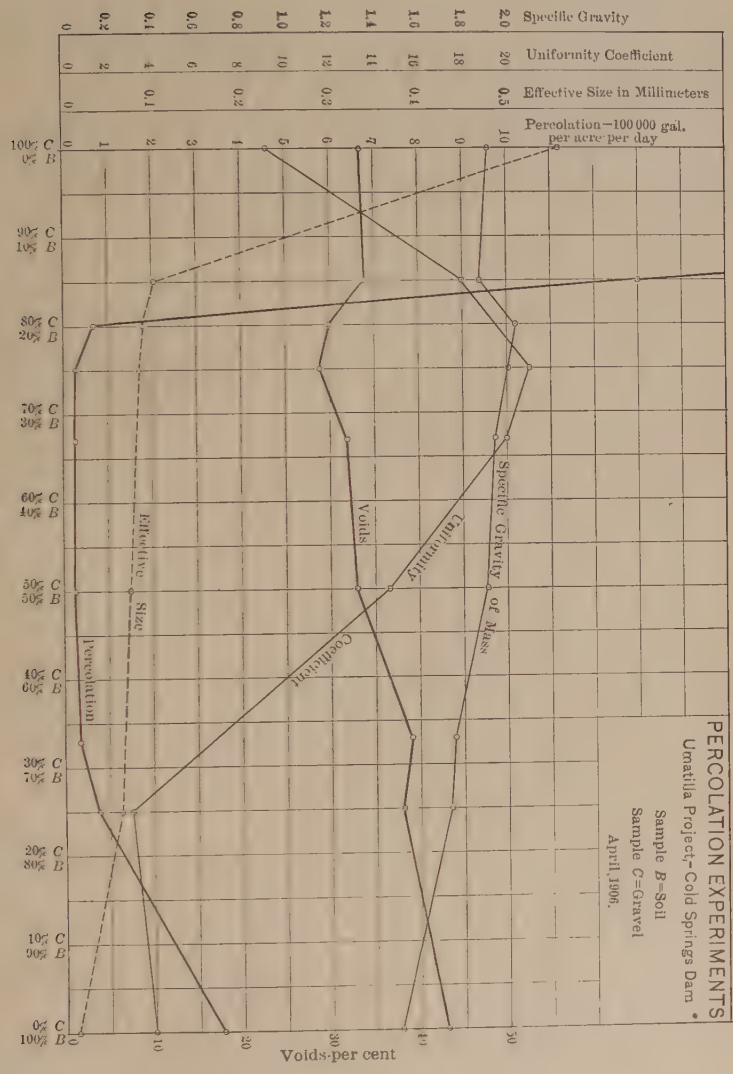

PERCOLATION EXPERIMENTS
Umatilla Project-Cold Springs Dam
Sample B=Soil
Sample C=Gravel
April, 1906.
Specific Gravity
Uniformity Coefficient
Effective Size in Millimeters
Percolation—100 000 gal. per acre-per day
Voids
Uniformity Coefficient
Specific Gravity of Mass
Effective Size
Percolation
100% C 0% B
90% C 10% B
80% C 20% B
70% C 30% B
60% C 40% B
50% C 50% B
40% C 60% B
30% C 70% B
20% C 80% B
10% C 90% B
0% C 100% B
Voids-per cent

FIG. 2.

and width up the hillsides until it is 6 ft. deep and has a bottom width of 10 ft. at the ends of the dam. To retard the flow along the plane of contact with bed-rock, a thin cut-off wall, 7 ft. high, was constructed on the center line of the trench, across the cañon, reducing in height up the side-hill. Five additional walls were built on the north hillside where the rock was exposed.

Provision was made for drainage by a gravel-filled trench along the entire down-stream toe of the dam with tile drain, and a network of additional trenches under the high portion of the dam, consisting of a parallel trench 120 ft. up stream from the water toe, with cross-trenches every 100 ft.

Slope Protection.—The water face of the dam is covered with 12 in. of pit gravel and 24 in. of rock rip-rap. The down-stream face is covered with 12 in. of rock rip-rap principally to prevent the gravel from being displaced and as a protection against wind action.

Inlet Structure.—The feed canal enters the reservoir near the south end of the dam. No rock could be found over which to drop the water, and a protective structure therefore became necessary. This consists of two continuous reinforced side-walls, with bottoms stepped and tops sloping, with cross-walls forming water-pockets, floored with concrete and covered with wooden wearing floors, the water spilling over each cross-wall and dropping into the water-cushion formed by the next cross-wall. There are nine water-steps, with a vertical drop of 6 ft. each, the lowest cross-wall having its top at low-water elevation. At the top of the structure the side-walls spread apart to accommodate a steel-plate weir with a 24-ft. crest.

This drop structure was built in deep excavation, it being left to the incoming water to overflow through a training furrow and cut its own channel to low water in the reservoir. This was partly accomplished in the course of three weeks by a flow varying from 30 to 100 sec-ft. Several steps still remain buried, owing to the grade assumed by the bottom of the cut. The sides of this cut stood at first vertical, but have gradually slumped, and are tending to a uniform slope.

About $\frac{1}{4}$ mile up stream from this structure, following the canal line, and where the canal alignment crosses the extended center line of the dam near its south end, a concrete gate structure has been built, with four arched openings, 6 ft. 6 in. wide in the clear, with hinged floating gates to prevent the reservoir water, when raised above the

spillway height by cloudbursts, from running back into the canal and overtopping its banks.

Flash-boards are also provided for backing the canal water over a concrete crest in the canal bank some distance up stream into a lined channel, affording direct connection between the inlet and outlet canals, and the means of by-passing the reservoir.

Outlet Conduit and Gate-Tower.—The outlet conduit and gate-tower are located on bed-rock on the south side of the cañon, with outflow elevation at 560 ft., or approximately 17 ft. above the cañon bottom. There are two 4 by 4-ft. Coffin sluice-gates, placed in series, one at the outside of the gate-tower and the other at the inside, the former being an emergency gate.

The conduit is founded on rock, and there are cut-off collars at intervals of 30 ft., the conduit discharging into a canal cut in the rock for about 200 ft., below which it continues in earth. An overflow spillway is provided in the canal bank, cut in the rock section, the spill crest having concrete guide-walls to the bottom of the cañon. The outlet canal capacity is 225 sec-ft. The gate-tower at the upper end of the conduit stands near the water toe of the dam. It is built of concrete, slightly tapering, and is connected with the top of the dam by a steel footbridge.

Spillway.—On the hard clay bed on the north hill shoulder, a spillway was built, with a crest length of 330 ft. and a crest elevation of 8 ft. below the top of the dam. At each end there are abutment walls, with cut-off walls reaching into the body of the dam. The water, after spilling over the crest, slides down an inclined concrete-paved slope and enters a concrete-lined channel with a general direction parallel to the overflow crest and at right angles to the dam. This channel is excavated in thorough cut past the line of the dam, through hard clay and fine sand formation, and terminates in hard rock at an elevation of 40 ft. above the cañon bed, the end being provided with a heavy cut-off wall extending well into the rocky sides.

The entire spillway channel has a concrete lining built in disconnected sections to 10-ft. squares, with joints underlaid by concrete cut-off walls. Where it passes through sand formation, it is reinforced with $\frac{3}{8}$-in. bars laid in 12-in. squares, and has weep-holes on the dam side. The capacity of the spillway, with 3 ft. of water over the crest, is assumed at 5 900 ft., at which point the dam will have a freeboard

of 5 ft. independent of a super-elevation of 2% of the height for possible settlement.

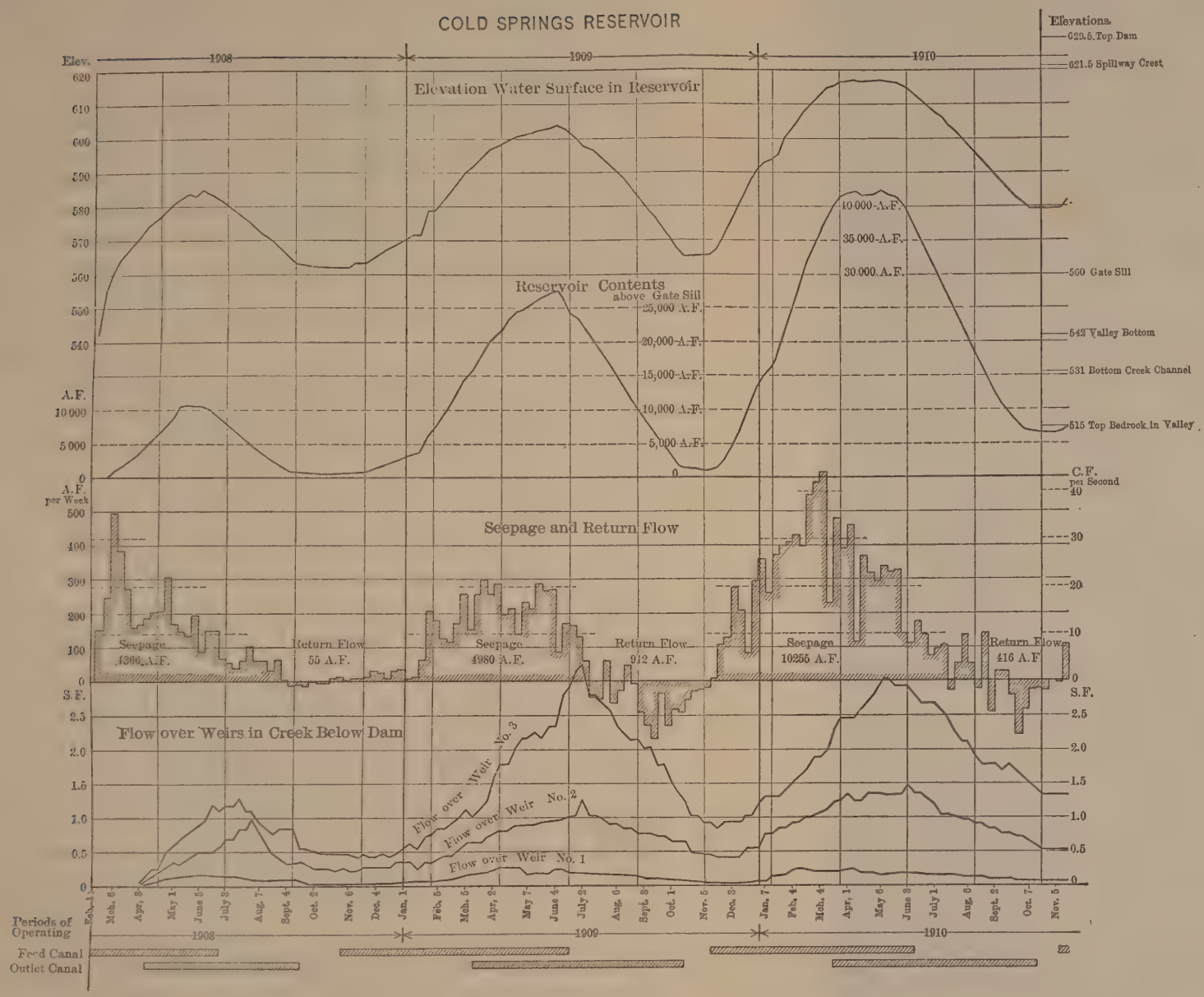
There had been no observed run-off from the drainage area of the reservoir prior to the commencement of the construction of the dam. During January, 1907, when the construction trestle was being erected, a heavy rain occurred, falling on 10 in. of snow, with the ground beneath frozen by previous cold weather. Three floods followed in rapid succession, discharging in the aggregate 15 000 acre-ft., and peaking for 3 hours at 6 000 sec-ft. Approximately 10 000 cu. yd. of material were eroded within the lines of the dam base. Floods of this type can only occur in the winter, at which time the reservoir can be but partly filled. In summer, cloudbursts may be expected, but their aggregate run-off is usually small—the disastrous Heppner cloudburst of 1903, for instance, resulting in an aggregate flow estimated at only 1 600 acre-ft.

With a spillway capacity of 5 900 sec-ft. at 3 ft. depth on the crest, the reservoir has a capacity, above the crest, of 7 000 acre-ft. The floods of 1907, above mentioned, were entirely unlooked for, but fortunately did no damage other than eroding part of the dam foundation. The creek channel would not ordinarily accommodate in excess of a few hundred second-feet, and the heavy growth of sage-brush on its banks indicated absence of any recent large flow. This is one of the many illustrations of the danger involved in estimating maximum floods from drift lines or other surface appearances.

Method of Construction.—The design called for the handling of approximately 490 000 cu. yd. of gravel from the gravel pit in the north cañon side, about ½ mile below the dam; for the excavation of 191 000 cu. yd. of loam or subsoil, to be obtained mostly from the slopes within the reservoir; and for about 36 000 cu. yd. of rock pitching, to be placed on both slopes.

The gravel was excavated with a 70-ton, Model No. 60, Marion steam shovel, with 2½-cu. yd. bucket. The rolling stock consisted of fifty 4-yd. side-dump cars, and five 16-ton American locomotives running on a 3-ft. gauge track of 35-lb. rails. The track system is shown on Plate VIII, the average distance of gravel haul being 2½ miles, and the maximum grade being 1¼ per cent. The gravel in the pit rose to from 30 to 60 ft. above the shovel, and was in places overlaid with considerable soil, rendering it necessary to watch the proportions of soil and gravel as they came on the cars to the dump.

PLATE IX.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXIV, No. 1205.
HENNY ON
TWO EARTH DAMS
OF THE U. S. RECLAMATION SERVICE.



The shovel was served by four gravel trains, of from nine to twelve cars each, which handled on an average, including moving and delays, about 1 600 cu. yd. per shift of 8 hours, the output per shift occasionally exceeding 2 200 cu. yd. The total rise from the steam shovel to the top of the dumping trestle was 65 ft., and the coal consumption per locomotive, for 1 149 shifts of 8 hours, averaged 2 400 lb., coal being obtained from the Kemmerer mines in Wyoming.

The gravel was delivered on the dam by dumping from a trestle, 65 ft. high, built across the cañon, with its center line about 60 ft. down stream from and nearly parallel to the center line of the dam. The entire trestle came within the 100% of the gravel section, and the posts were left buried in the gravel, but all bracing was removed as the work progressed.

The fine subsoil or loam was obtained from various sources, as follows:

From surface layers overlying the gravel in the gravel pit	100 000	cu. yd.
From borrow-pits on the side-hill, up stream from the dam	108 600	" "
From the feed canal and trenches	40 700	" "
From the spillway channel	17 700	" "
Total	267 000	cu. yd.

The loam from the gravel pit was handled in the same manner as the gravel. The loam from the borrow-pits was handled by wheel-scrapers up to distances of 500 ft. (22 300 cu. yd.), and by dump-wagons loaded by an orange-peel excavator for greater distances up to 2 000 ft. (86 300 cu. yd.). The loam from other sources was moved by scrapers, only that portion excavated from trenches, etc., being used which was found suitable, the remainder being wasted.

The loam was delivered first, in its proper proportions, and spread by a road scraper, after which gravel was spread over it, being scraped by Fresno scrapers from the foot of the gravel dump near the trestle. The materials were mixed at first with disk-harrows and subsequently with cultivators, the points of which scraped over and into the top of the previously mixed and rolled layer, after which the material was watered and rolled, producing a final layer of from 4 to 5 in. compacted into a hard mass which required picking to excavate. Constant

watch was kept of the thoroughness of the mixing process, by excavating test-pits. It was found impossible to secure complete mixing at all times. Unmixed gravel was nowhere found, but in some places fine streaks of loam had been left unmixed with gravel, in spite of every effort to avoid it.

The gravel in the 100% gravel section was not rolled to a large extent and was not watered. When the gravel dump had reached the height of the trestle, the track was raised by grading up, until the full height of the dam was attained.

The rock for slope pitching was obtained from the rocky basalt bluff on the north side of the cañon, a short distance below the dam. It broke up in fragments from 1 cu. ft. in volume down. A part of it was loaded by an orange-peel excavator, but most was handled from wheel-barrows into dump-cars. It was hauled by rail, dumped on the slopes from the dump-cars, and sloped by hand. The total required was 36 000 cu. yd.

The construction of the auxiliary structures contained no elements of special interest.

The work on the installation of the plant was commenced in December, 1906. The first gravel was dumped in March, 1907, and the dam was completed in June, 1908.

Shrinkage.—The design of the dam calls roughly for one-third of the mass 100% gravel, one-third 50% gravel, and the remaining third 67%, 75%, and 80% gravel. Laboratory tests indicated a shrinkage of about 10% for the various mixtures, and from this it was figured that there would be required 490 000 cu. yd. of gravel and 191 000 cu. yd. of loam, or a total of 681 000 cu. yd., to make the 637 000 cu. yd. of compacted dam.

The actual quantity of gravel excavated corresponds closely to the estimate, but the quantity of soil handled was 76 000 cu. yd. in excess of that figured. This large excess must be principally attributed to the difficulty of keeping the soil and gravel apart in the gravel pit, and may be partly due to the occurrence of volcanic ash or dust in the delivered soil, which may not have assisted in swelling the quantities. The proportion of loam in the gravel, where it came unavoidably mixed with gravel from the gravel pit, may have been under-estimated, and it is quite probable that much of the gravel, which from all appearances contained no soil, may have held proportions of from 5 to 10 per

FIG. 1.—GRAVEL BANK ON NORTH SIDE OF CAÑON, BELOW COLD SPRINGS DAM.

FIG. 2.—BASE OF DAM, CUT-OFF TRENCH, CONSTRUCTION TRESTLE, ETC., COLD SPRINGS DAM.

cent. While the excess soil has added to the cost, it can hardly be deemed injurious, as regards the drainage qualities of the gravel or its stability, and it has also added to its mass weight.

Cost.—The total cost of the dam, arranged by its principal features, is shown in the following tabulation:

Main dam		$364 140
Auxiliary structures:		
Inlet works	$16 140	
Outlet works	19 710	
Spillway	35 010	
		70 860
Preliminary engineering		5 000
Total, 49 000 acre ft., at $8.98 per acre ft.		$440 000

General administration, engineering and supervision, other than preliminary engineering, are included in the above figures.

The principal details of the cost of the main dam are as follows:

Embankment (yardage on basis of excavation measurement):				
Material from gravel pit: Gravel	490 000 cu. yd.			
Earth	100 000 " "			
	590 000 cu. yd.	at	$0.385	$227 020
Material from borrow-pits, spillway, and trenches:				
Earth, 86 300 cu. yd., orange-peel excavator, 36.1 cents				$31 120
80 700 " " wheel scrapers, 19.4 "				15 630
167 000 cu. yd., average, at $0.28				46 750
Rip-rap:				
From quarry...32 500 cu. yd., at 1.46				47 480
" trenches. 3 400 " " (charged to excavation).				
Excavation:				
Earth from trenches, spillway, etc., temporarily or permanently wasted	34 400 cu. yd.		0.293	10 060
Hardpan or loose rock	6 600 " "		1.37	9 020
Solid rock	4 000 " "		2.84	11 340
Concrete cut-off walls	327 " "		13.79	4 510
Drainage, including all work to July 1st, 1910				7 960
				$364 140

A further analysis of the two principal items may be of interest:

Embankment: Material from gravel pit, 590 000 cu. yd.
Steam-shovel excavation: Rail haul, average distance $2\frac{1}{4}$ miles, average rise 65 ft.

Cost per cubic yard, measured in excavation:

Steam shovel		3.6 cents.
Transportation: Railroad operation	5.5 cents.	
Track maintenance	1.7 "	7.2 "
Work on dam: Scraping, spreading, and mixing	8.2 "	
Sprinkling	0.2 "	
Rolling	0.5 "	
Water supply	0.7 "	9.6 "
Plant depreciation	10.4 "	
Plant maintenance	0.4 "	10 8 "
General supplies		1.8 "
Camp, shops, warehouses		1.2 "
Cleaning up, transfer of plant		0.6 "
Engineering, administration and general expenses		3.7 "
		38.5 cents.

Embankment: Material from borrow-pits, 86 300 cu. yd., orange-peel excavator, wagon haul, 500 to 2 000 ft.

Cost per cubic yard, measured in excavation:

Excavation		13.1 cents.
Wagon haul		6.9 "
Sprinkling		0.4 "
Rolling		0.7 "
Water supply		0.8 "
Plant depreciation	8.5 cents.	
Plant maintenance	0.4 "	8.9 "
General supplies		1.1 "
Camp, shops, and warehouses		1.2 "
Cleaning up, transfer of plant		0.5 "
Engineering, administration and general expenses		2.5 "
		36.1 cents.

FIG. 1.—CUT-OFF TRENCH ACROSS VALLEY BOTTOM, COLD SPRINGS DAM.

FIG. 2.—TRENCH FOR OUTLET CONDUIT, SHOWING CHARACTER OF ROCK, COLD SPRINGS DAM.

The cost of gravel and earth for the main embankment, based on bank measurement, is higher than the foregoing figures indicate, by reason of heavy shrinkage, and is shown as follows:

Excavation measurement.

Material from gravel pit	590 000 cu. yd.,	at 38.5 cents	$227 020
" " borrow-pits, etc.	167 000 " "	at 28.0 "	46 750
Total	757 000 cu. yd.	at 36.2 cents	$273 770
Embankment measurement	637 000 cu. yd.	at 43.0 cents.	

The prices paid for labor and fuel were as follows:

Common labor	$1.60 to $2.40 for 8 hours.
Teamsters	2.40 to 2.60 " " "
Teamsters, with team	4.50 " " "
Steam-shovel engineers	6.20 " " "
Cranesmen	4.00 " " "
Locomotive engineers	3.60 " " "
Carpenters	$3.60 to 4.00 " " "
Coal, per ton, on work	8.63

Behavior of Reservoir Since Completion.—Water was first admitted into the reservoir on February 19th, 1908, and reached its maximum height for that year on June 5th, at Elevation 584.4, and its minimum height on October 20th, at Elevation 561.8. Water was again admitted on November 5th, 1908. In 1909 it rose, on June 4th, to 604.1, and dropped, on October 15th, to 564.7. In 1910 it rose, on March 25th, to 616.2, and, on May 11th, to 616.7, standing at that time within 4.8 ft. of the spillway crest, with the reservoir containing 42 000 acre-ft.

From the time water was first admitted, measurements have been made of the inflow through the feed canal and from Cold Springs Creek at times of flood, of the outflow through the outlet canal, and of the evaporation and rainfall, which have yielded some interesting information as to seepage losses, as shown graphically on Plate IX.

Close watch for first evidences of seepage below the dam was also kept. The drain tile in the lower toe of the dam, serving the portion to the north of the outlet conduit and including the highest part of the dam, discharges into a small concrete basin in the old creek bottom, which is provided with a weir (Weir No. 1, on the plan of the dam, Plate VIII). The first water came from this drain

on April 2d, 1908, with the reservoir water at 571.5, or 27.5 ft. above the valley bottom. The maximum flow during 1908 was 0.17 sec-ft. During 1909 it was 0.26 sec-ft., and during 1910, 0.26 sec-ft.

In the flat below the toe of the dam, for a distance of about 1 000 ft., ground-water was observed to be gradually rising, and water began to ooze from the soil banks on either side of the meandering creek channel. This condition gradually extended downward for a distance of ½ mile. Weirs were maintained at two points in the creek channel, Weir No. 2 about 500 ft., and Weir No. 3 about 3 000 ft. down stream from the dam; and the flow, as measured, is shown on Plate IX.

It should be understood that the outlet canal skirts the south cañon side, and after leaving the rock adjoining the dam, runs through fine, sandy material, seepage below the hillside being evident in many places. This contributed to the flow, as measured in the creek weirs, to an extent which it has not been possible to determine definitely. The time during which the outlet canal was operated is shown at the bottom of the diagram, Plate IX, from which it will be noted that this flow is stopped in the fall of the year after the reservoir has been practically emptied; so that the effect of cessation of flow in the canal and that of the lowering of the water surface in the reservoir are combined in the reduced flows measured at the weirs. From the diagram it will be seen that the flow at Weir No. 2 reached maxima for the three successive years of 1.0, 1.26, and 1.46 sec-ft., while that at the lower creek weir shows maxima of 1.28, 3.27, and 3.03 sec-ft., respectively, these flows peaking about 4 weeks after the highest water occurred in the reservoir.

The portion of the dam to the south of the outlet conduit is served by a drain which discharges into the conduit. This did not show any flow during the years 1908-09, but in 1910 the flow was 0.08 sec-ft. during May, and, with the lowering of the water, this has again dried up. The lower end of this drain is at Elevation 575, or 29 ft. and 42 ft., respectively, below high water in 1909 and 1910. When moisture showed in the low places in the flat immediately below the dam in 1908, it was deemed advisable to lay additional drainage pipes, and, during the following winter, and also during the winter of 1909-10, a more complete system, consisting of a main 12-in. drain and cross-lines of 3-in. drains, was constructed, and has been entirely effective in maintaining the surface in a perfectly dry condi-

FIG. 1.—SPILLWAY, COLD SPRINGS DAM.

FIG. 2.—CONSTRUCTION TRESTLE BURIED IN GRAVEL DUMP, COLD SPRINGS DAM.

tion. Some drains have also been laid at the foot of the hillside below the outlet canal, to intercept seepage from this apparent source.

The course of seepage water from the reservoir is uncertain. It is not improbable that a small quantity of water passes through the body of the dam. The flow from the drain serving the highest portion of the dam carried about 0.25 sec-ft. with the reservoir water at 616.8, or 76.8 ft. above the valley bottom. The appearance of oozing water along the sides of the creek channel, for a considerable distance below the dam, indicates that seepage water may travel through seams in the underlying rock, or entirely around the outer slopes of the rock. It is significant that lower portions of the flat, separated from the dam by the meandering creek channel, show water well above the creek bottom, indicating a roundabout course of this seepage water.

The exposed rock surface of the bluff on the north side of the cañon immediately below the dam showed no seepage until 1910, when a small spring made its appearance near the foot of the bluff, close to the toe of the dam, attaining a maximum flow during the season of 0.044 sec-ft. The interior of the outlet conduit was carefully examined, and while some moisture was in evidence, no dripping leaks were found where the conduit passes through the water-tight section of the dam. One small squirting leak, the size of a fine straw, was found near the lower end of the conduit where it passes through the 100% gravel section of the dam immediately above the highest cut-off wall, the pressure probably being due to the head accumulated by this wall, which makes tight connection with the tunnel rock trench in which the conduit was built.

As regards seepage under the dam on the south hillside, it has been stated that the maximum flow measured from the drain was 0.08 sec-ft. during 1910; the only further evidence of seepage along this part of the dam was a moist spot near the toe of the dam at about Station 17, where the elevation of the ground is about 600, or 17 ft. below the highest water in 1910. An investigation will be made as to whether this seepage may have been due to a break or obstruction in the drain pipe. Such break was found near Station 27, in the highest part of the dam, in April, 1909, due undoubtedly to insufficient bedding of drain pipe. Just previous to this break, the drainage water from this line of tile, which had always been running crystal clear, showed a small amount of very fine sand, while the delivery

dropped from 0.25 to 0.19 sec-ft. Repairs were made during the following fall, at a time of low water in the reservoir. The down-stream slope of the dam has at all times been perfectly dry, nor is it conceivable that water would show at an elevation higher than the ground adjoining.

The diagram on Plate IX is especially interesting in that it shows the reservoir seepage losses determined by striking a balance between measured inflow and precipitation on the one hand, and measured outflow and evaporation on the other. The rate of loss at first filling was about 30 sec-ft., which quickly dropped to 20 sec-ft., and then continued to drop slowly during the first season until a slight return flow was measured. The maximum rate of loss during 1909 was 17 sec-ft., when the submerged area measured 1 003 acres, and in 1910 this rate rose to 34 sec-ft., at which time the water-covered area in the reservoir was 1 310 acres.

The amount of seepage during filling is dependent not only on the area submerged and the voids in the underlying soils, but also on the rate of filling. The higher this rate, the greater will be the new area submerged in a given time, and the steeper will also be the water gradient in the adjoining area. It is possible that this fact may account for the apparent anomaly of the seepage curve peaking several weeks prior to the dates when highest water in the reservoir was reached. The rate of filling is indicated by the steepness of the capacity curve, and it will be noted that the summits of the seepage curve coincide fairly with the breaks in the upward slope of the contents curve, showing a reduction in the rate of filling.

The dependence of seepage on the rate of filling is also indicated by the marked difference in seepage for the same water elevations under conditions of filling and of emptying the reservoir. For instance, on August 10th, 1909, return flow, during emptying, commenced with the reservoir water at 589, while previously, on February 26th, 1909, with water at the same elevation, the seepage loss during filling was 14 sec-ft. Similarly, in 1910, the return flow commenced on August 27th, with the reservoir emptying and with the water at 591, while on December 30th, 1909, with the water at the same elevation and the reservoir filling, the seepage loss was 19 sec-ft.

The totals of the losses from the reservoir for various periods are shown in Table 2.

FIG. 1.—TOP AND DOWN-STREAM SLOPE, COLD SPRINGS DAM.

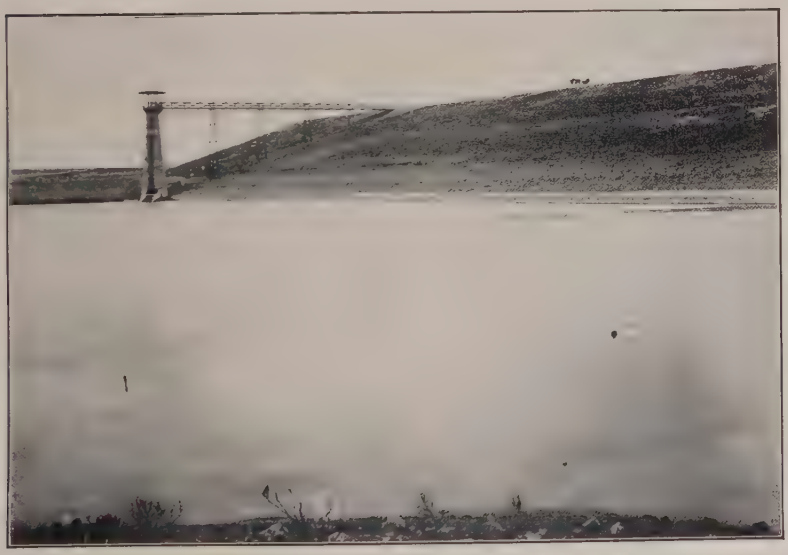


FIG. 2.—UP-STREAM FACE AND GATE-TOWER, COLD SPRINGS DAM.

TABLE 2.—Losses from Cold Springs Reservoir.

Period.	Total loss, in acre-feet.	Evaporation loss, in acre-feet.	Seepage loss, in acre-feet.	Maximum area submerged, in acres.
February 19th, 1908.....				
November 1st, 1908.....	6 717	2 384	4 333	659
November 1st, 1909.....	8 311	4 254	4 057	1 060
November 1st, 1910...	14 916	5 122	9 794	1 836
	29 944	11 760	18 184	.....

The evaporation loss was determined by applying the evaporation, measured each week in a floating pan, to the average reservoir surface for that week. This method leads to too small results, because the evaporation proceeds from the marginal areas as well as from the water surface, and because the actually exposed reservoir surface at times of wind, when evaporation is most rapid, is increased by the formation of waves to a far greater degree than is the case in a pan. Therefore, the resultant seepage losses as given are correspondingly too large, the writer having been unable to find a satisfactory basis for applying corrections.

The only evidence of escaping seepage water was in the creek channel below the dam, which averaged, as measured on Weir No. 3, for the three periods given in Table 2, 0.7, 1.4, and 1.8 sec-ft., respectively, resulting in an aggregate visible flow of 2 630 acre-ft. Assuming that one-fourth of this may be charged to outlet canal loss, 1 970 acre-ft. may have come from the reservoir as visible flow and the remainder, 18 184, less 1 970, or 16 214 acre-ft. of reservoir seepage must have been stored in the ground during the period of 2 years and 10½ months. Comparing this with the maximum area covered (1 386 acres), the loss is about 11.8 ft. in depth per acre, sufficient to fill soil having 45% voids to a depth of 26 ft. As the depth to bed-rock may average more than this, and especially as the water movement must be largely horizontal, the losses above recorded do not seem surprising. It is probable that losses of this character will continue for several years, until some average water-table in the surrounding soil shall have been established, after which the loss during the early season will be largely offset by return flow on emptying the reservoir.

Settlement.—The dam was finished with a super-elevation of 2% of the height, as previously stated, to take care of possible settlement. Bench-marks established on the concrete bridge piers at the time of construction and at points on the water slope where the dam is highest, show a settlement to date of about ¼ per cent. A series of bench-marks was established on top of the dam on January 2d, 1909, after a depth of 41 ft. of water had stood against the dam during the previous season. These bench-marks show a subsequent settlement, varying from 0.1 to 0.45% of the height, the maximum settlement at any point being 0.366 ft., where the dam is 84 ft. high.

The dam design was approved by the Director and the Chief Engineer of the Reclamation Service, and by A. J. Wiley, M. Am. Soc. C. E., as Consulting Engineer. The designs were prepared and the work executed under the general supervision of Mr. E. G. Hopson, Associate Supervising Engineer, and the writer. The project was in immediate charge of John T. Whistler, M. Am. Soc. C. E., and the engineering and construction work at the dam was in charge of Mr. B. H. Davis. For the last two years the project has been in charge of H. D. Newell, Assoc. M. Am. Soc. C. E., who has directed the later work and measurements.

Conconully Dam.

This dam is lower than the Cold Springs Dam and contains less material, but is of comparative interest as it was built by the hydraulic process. It is located in Northern Washington, on Salmon Creek, a branch of the Okanogan River, which empties into the Columbia River. The dam is part of the Okanogan Project of the United States Reclamation Service, intended for the irrigation of 10 000 acres of land along the west bank of the Okanogan River, between the Towns of Okanogan and Riverside. The land lies in benches above the Okanogan River, which has too flat a grade to serve as a source of supply. The water, therefore, is derived from the Salmon Creek, and as the midsummer flow is very small, the storage of flood flow was necessary.

Part of the needed storage (1 800 acre-ft.) is found in utilizing Salmon Lake, situated in a side cañon just above the main reservoir. The main storage was created by the Conconully Dam, the principal features of which are as follows:

Greatest height above bottom of creek channel	66	ft.
Width of valley on center line of dam.	815	"
Length of crest of dam	1 010	"
Top width	20	"
Length of spillway	180	"
Up-stream slope: upper portion $2\frac{1}{2}:1$, lower portion	3:1	
Down-stream slope	2:1	
Volume of dam	351 500	cu. yd.
Capacity of reservoir above outlet gates	13 000	acre-ft.
Required capacity of outlet works	100	sec-ft.
Area of reservoir at high-water line	460	acres.
Drainage area above dam	113	sq. miles.

The dam is intended to store the waters of the creek on which it is located, and is therefore independent of a feed canal. The drainage area above the dam is of distinctly mountainous and forested character, much of it being in forest reserve. The flow from Salmon Creek varies from 10 to 20 sec-ft. during 8 months of the year. The flood months are April, May, June, and July, during which the flow usually ranges between 100 to 300 sec-ft. The maximum flow recorded since 1904 is 577 sec-ft.

Reservoir Site.—The dam is 15 miles above the mouth of Salmon Creek and immediately below the junction of the two forks called the North and the West Forks, as shown on Fig. 3. The water in the reservoir fills a basin with a high-water area of 460 acres, the water backing up to the limits of the Town of Conconully.

The reservoir is surrounded by mountains on all sides except to the east where an extensive gravel bar exists, which is probably a glacial moraine. Without this bar, the water from the reservoir would flow in an easterly direction down Scotch Creek. The bar at its southern extremity rests against the mountain slope which farther down forms the east abutment of the dam. The width of the bar at the spillway crest elevation is about $\frac{5}{8}$ mile. On its eastern slope small springs exist, which give rise to Scotch Creek. Water first appears at an elevation above that of Salmon Creek on the west side of the bar, although

most of the springs are at a lower elevation. It is possible that these springs may result in part from seepage from the creek, but it is considered more likely that they find their origin in drainage from the mountain side to the south. In considering the possibility of seepage

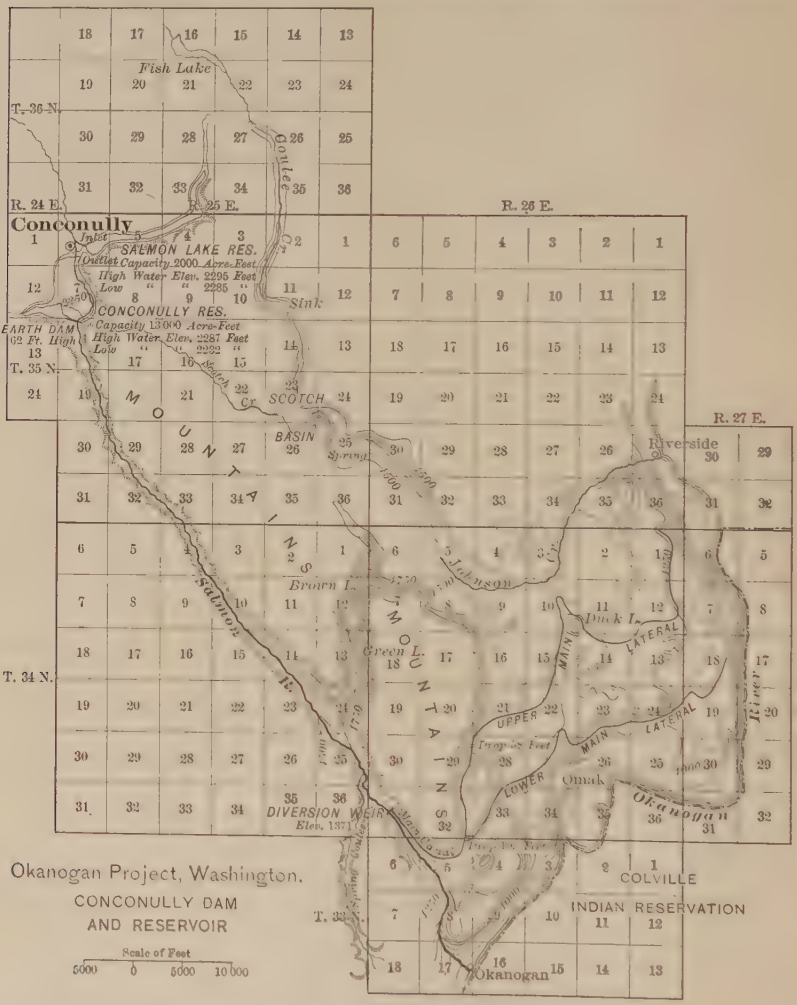


FIG. 3.

through this bar, from the rise of water in the reservoir, it was found that the average water grade with reservoir full would be about 1 : 165, and, aside from the fact that the reservoir side of the bar is blanketed with mixed material (including some surface loam), and that the

gravel is principally made up of sand, it was believed that the slight water slope precluded any possibility of material losses.

The flow of Scotch Creek has been measured to determine possible increases, and if any should take place, it will be easily possible to divert it for the irrigation of the lower lands. The reservoir water is released into Salmon Creek and picked up at a diversion dam about 11 miles down stream.

The reservoir bottom consisted of meadows, and was covered with thick brush, which has been slashed and burned. The surface soil is a black sandy loam for a depth of 4 or 5 ft., below which the sand gradually becomes cleaner, deeper layers occasionally showing coarser material, the rock being at unknown depth both in the reservoir bottom and at the dam site, where soundings were continued to a depth of 95 ft.

The dam is at the outlet of the valley, where it contracts to a width of about 1 200 ft., the gap being further reduced by a low mountain spur from the west, leaving an opening, about 1 000 ft. long, to be closed by the dam, the spur serving the purpose of a spillway by being cut down for a width of 180 ft. to the proper elevation.

The formation of the hillsides is a close, heavy granite, but in the above mentioned spur this granite merges into limestone of a rather soft and seamy character.

Design of Dam.—The total height above the valley bottom to which the water was to be raised is about 50 ft. The foundation could not be made water-tight, and the design simply aimed at a retardation of the percolation by providing a broad base to the dam due to flat slopes, and by driving a row of sheet-piling, running across the valley, joining the rock slopes at both ends.

Materials Available.—The following materials were available for dam construction:

Fine sandy loam near the surface, in the valley bottom, principally to be found up stream from the dam;

Gravel and sand from the gravel bar to the east of the reservoir, at a distance of from 2 000 to 5 000 ft.;

Talus material on the west mountain side, just below the dam, consisting of sand and silt from the disintegration of the granite rock, mixed with angular rock fragments of sizes from a man's fist to a cubic yard.

The latter material was selected as that most suitable for the dam, in connection with the method of construction to be followed.

The location of the dam is quite remote, the nearest railroad station being Wenatchee, Wash., 105 miles distant, from which material can be shipped by river steamer to Brewster, whence the wagon haul over mountain roads to the work is 45 miles.

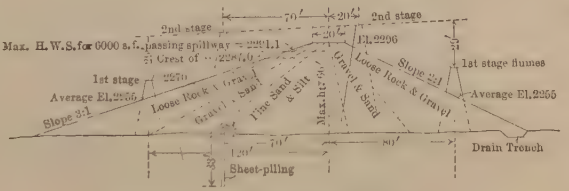
The country affords practically no local labor, and for machine repairs it would be necessary to send to Spokane, Seattle, or Portland. Thus it was desirable to depend as little as possible on mechanical equipment, and to utilize the local resources to their maximum extent. The use of the valley bottom material would have implied the installation of pumping machinery, in connection with either hydraulic or mechanical means of transportation, as the ground-water was close to the surface. Gravel and sand, obtainable from the gravel bar on the east side of the reservoir, would have necessitated the importation of a steam shovel, and probably railroad equipment. The material available on the steep mountain side above mentioned could be had at practically any elevation, and could be moved by the hydraulic process. Water was available in the West Fork of Salmon Creek by diversion through a 3-mile flume, the creek flow being 50 sec-ft. and more during April, May, and June, and gradually dropping to 10 sec-ft. in August and September.

The pit material, as revealed by numerous test-pits, appeared to contain a sufficient proportion of silt for producing a tight core. Consequently, the hydraulicking process was adopted, and the dam was designed in accordance therewith, the plan and sections being shown on Fig. 4.

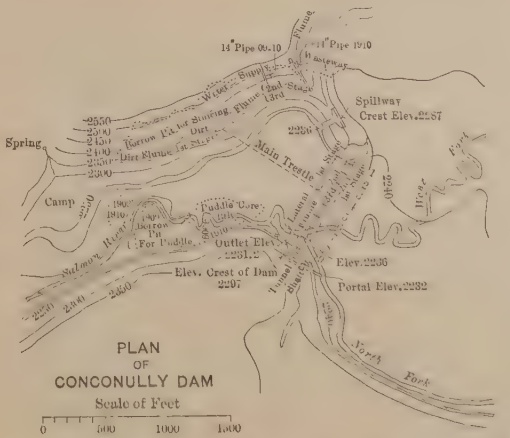
The character of the material hydraulicked in 1908 is best shown by the physical analyses (Table 3) after from 10 to 40% of material coarser than would pass a 7 by 13-in. screen had been eliminated.

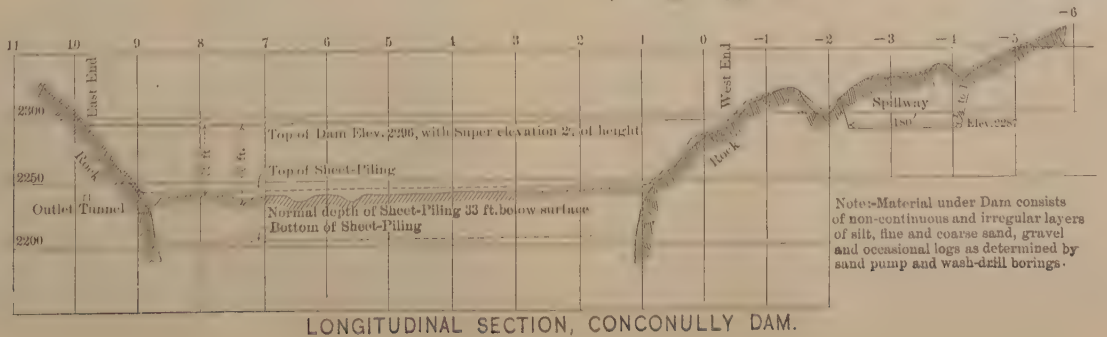
Much of the material hydraulicked in 1909 and 1910 was coarser than that from the 1908 pit No. 2. On the whole, the material was found to be far coarser than had been inferred from test-pit examinations.

It was considered desirable to place the core section as near forward in the dam as practicable so as to have it backed by the maximum quantity of more open material. As a result, the central plane through the core has a down-stream inclination. It was ex-



CROSS SECTION CONCONULLY DAM





LONGITUDINAL SECTION, CONCONULLY DAM.

FIG. 4.

pected that no difficulty would result from this position of the core, as it would be possible to keep the down-stream dumps at a higher elevation than those up stream and thus maintain the central pond at a point well forward toward the reservoir side.

TABLE 3.—Physical Analysis of Material Available for Conconully Dam.

Pass sieve.	Stop on sieve.	Percentage, by Weight:	
		Pit No. 1. Average of three samples.	Pit No. 2. Average of three samples.
7 by 13	2½-in.	15.8	40.65
2½	No. 4	27.2	32.69
No. 4	10	12.4	6.21
10	20	11.9	3.73
20	40	3.8	1.46
40	60	14.5	5.79
60	80	3.9	2.07
80	100	2.4	0.97
100	200	6.1	4.45
200	.....	2.0	1.98
		100.0	100.0

The core section connects with the side-hill by cleaning to bed-rock and excavating a rock trench in line with the inclined central core plain. A drainage trench was provided at the down-stream toe of the dam, filled with coarse material, hydraulicked in, connecting with the old creek channel below the dam.

The spillway has a crest length of 180 ft. and a crest height 9 ft. below the top of the dam, giving an approximate overflow capacity of 4 500 sec-ft., with 5 ft. freeboard on the dam, or about 40 sec-ft. per sq. mile of drainage area. It was cut into the west ridge, and was provided with concrete crest and channel lining, the point of discharge at the foot of the hill being 300 ft. from the down-stream toe of the dam.

The discharge from the reservoir is through a tunnel in the hard granite mountain side on the east, partly lined with concrete, in which two 36-in. common water-works gates were installed. These are set side by side, and operated through a vertical shaft connecting with the surface and well up stream from the center line of the dam. The tunnel was left unobstructed, and the water stored during the 1909 season was controlled by a temporary wooden gate in the shaft, the

FIG. 1.—SHEET-PILING AND FIRST-STAGE FLUMES, CONCONULLY DAM.

FIG. 2.—SECOND-STAGE FLUMES AND CENTRAL POND, CONCONULLY DAM.

final gates not being installed until the summer of 1909. The required discharge capacity is 100 sec-ft.

Construction.—Construction was commenced in the summer of 1907, during which year the following work was done:

Building 3 miles of feed-water flume, mostly on steep mountain sides;

Building a dirt flume, partly on mountain side, partly on trestle;

Driving and jetting 855 ft. of 6 by 12-in. triple-lap, tongued and grooved, sheet-piling, 36 ft. long, for a distance of 33 ft. into valley bottom;

Excavating 395 ft. of 8 by 9-ft. outlet tunnel, partly lined, through the east mountain side, and excavating a vertical shaft;

Partial excavation of the spillway gap in the ridge at the west end of the dam.

During the season of 1908, 97 000 cu. yd. of material were sluiced from the borrow-pits, and the spillway excavation was completed and lined with concrete. At the end of the season, second-stage flume trestles were erected.

During the season of 1909, 188 000 cu. yd. of material were sluiced from the borrow-pits, and the permanent gates were installed in the outlet tunnel.

During the season of 1910, the remaining 64 000 cu. yd. of material were sluiced, mostly from the second-stage, and partly from the third-stage flumes. The dam was completed during August, 1910.

The building of auxiliary structures presented no special points of interest, and the following remarks will be confined to the hydraulicking feature.*

The original supply flume had a capacity of 17 sec-ft. Its location is shown on Fig. 5. At the end of the 1908 season it was decided to increase this capacity to 26 sec-ft. Small storage reservoirs were built above the flume intakes to permit of concentration of the flow during the dry season for two shifts or one shift each day. The large boulders found in the pit had to be broken by blasting, or wasted, to prevent them from accumulating in the bottom. For this reason two pits were kept in operation alternately.

* A description by the writer of the details of the hydraulicking work during the year 1903 was published in *The Engineering Record*, April, 1909.

The feed-water was led down the mountain side through 14-in., No. 16, steel, slip-joint pipes, one line of pipe for each borrow-pit, and was used partly by the giant, which consumed from 1½ to 5½ sec-ft., through nozzles changed from 2 to 3½ in. in diameter, as required. The water was supplied under a pressure due to the difference in elevation of the feed-flume and the bottom of the pit, which difference ranged from 129 to 169 ft. for the first stage, and from 114 to 140 ft.

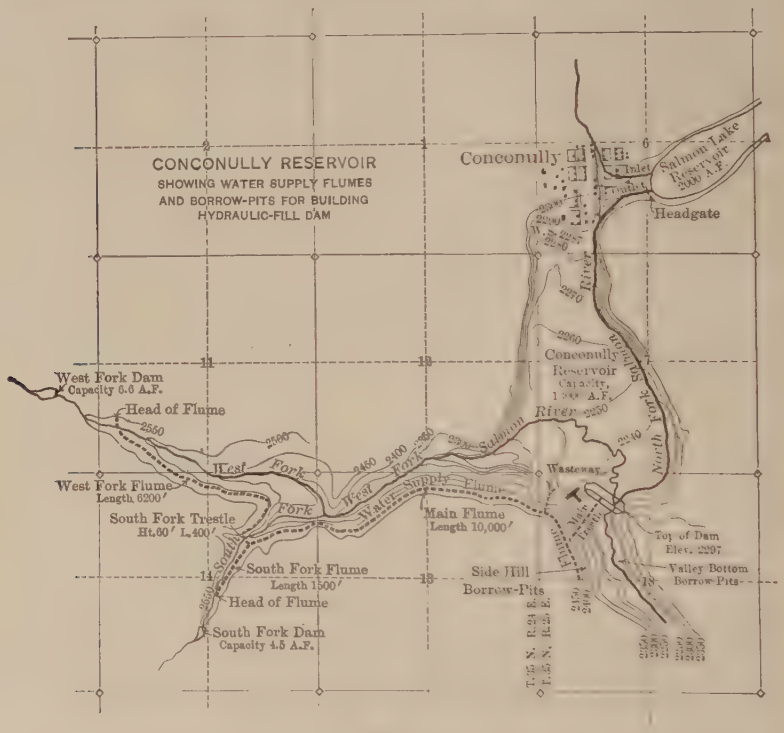


FIG. 5.

for the second and third stages. A flow of from 2 to 3 sec-ft. was delivered under pressure through a 4-in. pipe at the head of the borrow-pit dirt flume near its bottom, serving as push-water. The remainder of the available water was used as push-water at the point where the pit flume dropped its load into the main dirt flume. A small quantity of water, however, was allowed to enter the pit at its upper end on a level with the supply flume, in order to cause the fine upper material to slide in from above.

FIG. 1.—SECOND-STAGE FLUMES AND ARTIFICIAL CORE, CONCONULLY DAM.

FIG. 2.—THIRD-STAGE FLUMES, CONCONULLY DAM.

A 7 by 13-in. screen was used at the head of the pit flume during the 1908 season, to exclude large rock, but its use was discontinued after the water supply was increased.

The main dirt flume ran along the lower edge of the pits opposite the point on the dam equidistant from its ends, and then proceeded on a high trestle from the mountain side to the dam. On reaching the dam, the main flume connected on either side with two lateral flumes near the down-stream toe of the dam, and continued to similar flumes close to the up-stream toe. When the dam was built up to the elevation of the first lateral flumes, the main trestle was raised 29 ft., and was connected with a new flume laid along the mountain side, while the new lateral flume trestles were built closer to the center line of the dam. In the final finishing of the dam, a single flume was built on trestle near the center line.

The main dirt flume was built with wooden sides slightly inclined outward, and with a curved bottom of No. 10 mild steel with a 12-in. radius. The width at the top was 2 ft. 9 in., and the total depth 2 ft. 3 in. The velocity of the water ranged from 14 to 18 ft. per sec.

It soon became apparent that the angular rocks sliding on the bottom caused serious wear, and when 11 000 cu. yd. had been delivered, many holes had been worn within a strip in the center 6 in. wide, the steel higher up showing little wear. The flume was then given a flat wooden bottom 16 in. wide, and lined with No. 10 mild steel, which stood the wear far better.

At the end of the first season, when it was decided to increase the water supply from 15 to 26 sec-ft., the dirt flume was rebuilt to rectangular shape, with a bottom 30 in. wide and lined with ¼-in. high-carbon steel, and 27-in. sides lined for the lower 6 in. with No. 10 mild steel. The heavier and harder steel answered the purpose satisfactorily, and lasted through the delivery of 252 000 cu. yd. of material, showing serious wear only at the butt joints.

The flumes had 4% grades, except the short borrow-pit flumes, which had 8% grades, and the third-stage flumes for the finishing of the dam, which for part of the distance back had a 3% grade.

The material was discharged from delivery points at the dam in two rows of cones, forming ridges, the principal ridge being along the down-stream slope. By deflecting screens, gratings, spouts, and other means, the coarsest material was discharged, as far as possible, out-

ward, and the finer material inward, toward the pond maintained between the two ridges. The surplus water from the pond was drawn off on the reservoir side through flumes near the ends of the dam, which were alternately raised 6 in. at a time, the pond being maintained at a depth of from 12 to 18 in.

The material settling in the pond consisted of very fine sand and silt, the coarser sand and gravel coming to rest on the sloping sides of the pond, and the large rock dropping vertically and sliding down the cone slopes. The pond at first was quite wide, but, as elevation was gained, it narrowed up to such an extent that in spite of skillful handling, the sloping coarse sand layers would at times extend well into the puddle section and sometimes clear across it. Such layers were broken up by systematic stirring with paddles, but when this tendency to stratification became more marked and could not be satisfactorily prevented or counteracted, it was decided to introduce an artificial core with puddling material from other sources. The surface material in the valley below the dam, consisting of black, loamy sand, was well suited to form a core. This material was hauled in by scrapers on an up-hill road, dumped on a platform and washed into the dam through an 8-in. pipe. In order to insure against stratification across this core, two wooden diaphragms were built of 2 by 4-in. studding and 1-in. boards, which were first given a vertical position and made to step back in sections, so as to have their center plane correspond, as nearly as possible, with the center plane of the general core, but which were later built in a sloping position. Thus, while the material from the upper borrow-pits was hydraulicked in on the slopes, the core material was washed in through the 8-in. pipe between diaphragms. The artificial core was started at an elevation 14 ft. above the general base of the dam, in the late spring of 1909, and was continued about 39 ft. up to the high-water line. It contains in the aggregate 11 600 cu. yd., and, owing to the long haul of about 1 000 ft. on a 7% up-grade, and also to the necessity of using a large quantity of lumber for diaphragms, its cost was quite high.

The coarse rock, as dumped on the outer slopes, was of sufficient size to serve as rip-rap, but it did not prove possible to deposit it to final slopes by the use of water alone, and after hydraulicking was completed, it required a large amount of hand-work to obtain reasonably good slopes.

FIG. 1.—SIDE-HILL BORROW-PIT, CONCONULLY DAM.

FIG. 2.—MATERIALS BEING DUMPED FROM FLUME, CONCONULLY DAM.

As the quantity of rock found in the borrow-pits was larger than had been estimated, the relative quantity of rock on the water slope became sufficient to justify the steepening of this slope from 3:1, as had been originally designed, to $2\frac{1}{2}$:1 for the upper 26 ft. of the dam, as shown by the dotted lines in the central section on Fig. 4.

The dam was built with a super-elevation of 1 ft. across the valley, equivalent to nearly 2% of its height. In view of the hard pounding and washing which the material received in being dumped, and the prevalence of sand and gravel, this provision may seem excessive. During the progress of construction, however, it was found that as the load came on the base, considerable settlement occurred in the trestles, due apparently to a compacting of the fine loamy sand in the foundation. The amount of this settlement, extending over the last 560 days of construction, is shown on Fig. 6 by contour lines, the maximum settlement being 3.9 ft., and occurring along the old creek bed. In total volume, this settlement amounted to 15 500 cu. yd., and while it may have ceased on the completion of the dam, it was deemed wise to finish to the super-elevation above mentioned.

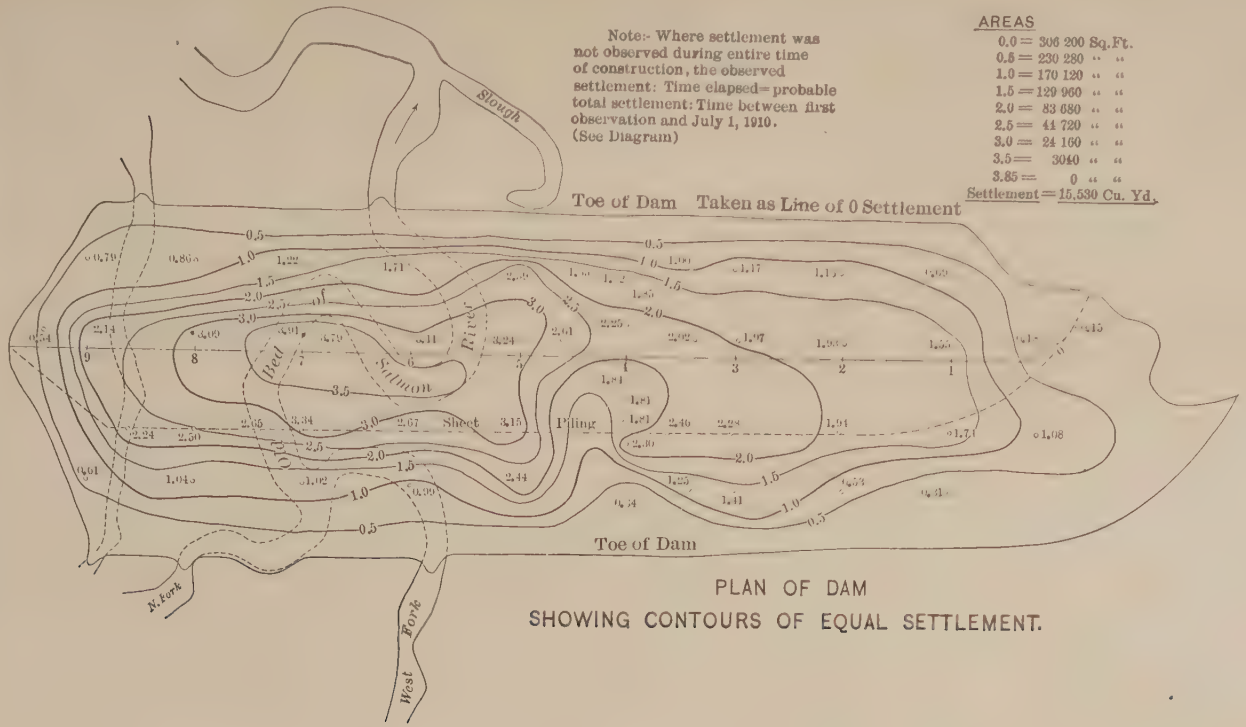
Careful watch was kept for possible evidence of a swelling up of the ground surface beyond the toes of the dam, but none was observed.

The degree of effectiveness of the sluicing for each season is shown in Table 4.

TABLE 4.—Sluicing Conconully Dam.

Season.	Hours actually sluiced.	Hours lost.	Average water supply.	Cubic yards from borrow-pits.	Average cubic yards per hour sluiced.	Average percentage of material carried.
April 22d–October 15th, 1908..	1 935	231	13.2	97 165	50.2	2.85
" 14th–November 13th, 1909.	2 892	564	20.2	188 300	65.1	2.42
March 31st–June 24th, 1910.....	1 154	297	22.3	64 900	55.4	1.87
	5 981	1 092	....	.....	....	....

The decreasing percentage of material carried by the water, in spite of increasing skill and larger water supply, is due to the increasing proportion of coarse rock in the borrow-pits. It will be observed that, of the total 7 073 hours of sluicing, 1 092 hours, or about 15%, were lost. This was due principally to clogs in the dirt flumes, and in part also to breaks in the feed flume, and to the necessity of replacing the dirt flume bottoms.


Note:- Where settlement was not observed during entire time of construction, the observed settlement: Time elapsed= probable total settlement: Time between first observation and July 1, 1910. (See Diagram)
AREAS
0.0 = 306 200 Sq.Ft.
0.5 = 230 280 " "
1.0 = 170 120 " "
1.5 = 129 960 " "
2.0 = 83 680 " "
2.5 = 44 720 " "
3.0 = 24 160 " "
3.5 = 3040 " "
3.85 = 0 " "
Settlement = 15,530 Cu. Yd.
Slough
Toe of Dam Taken as Line of 0 Settlement
Bed of Salmon River
Sheet Piling
Toe of Dam
N. fork
West Fork

PLAN OF DAM
SHOWING CONTOURS OF EQUAL SETTLEMENT.

Fig. 6.

The largest output in any one day was 181 cu. yd. per hour, at which time the water carried 5.3% of solids. The largest rocks carried in the flume, with 25 sec-ft. flow, weighed 250 lb.

The waste-water from the pond did not run out perfectly clear, but contained from 0.1 to 0.2% of silt, which sometimes rose to 0.5 per cent. The total loss of silt from this cause during the entire work is estimated at 20 000 cu. yd.

The segregating effect of the water caused an increase of dam measurement over borrow-pit measurement, possibly offset in part by a shrinkage where sand, gravel, and rock, with some silt, happened to be well mixed in the dump pile. The swell in volume during the first season was estimated at 12%, but, for the completed work, a lower percentage is computed, as follows:

Sluiced from side-hill borrow-pits.....		349 455 cu. yd.
Loss in waste-water.....		20 000 " "
Remaining in dam, pit measurement....		329 455 cu. yd.
Volume of dam above original surface....	336 000 cu. yd.	
Volume of dam below original surface....	15 500 " "	
Total volume of dam...	351 500 cu. yd.	
From valley pits.....	11 600 " "	
From side-hill pits, bank measurement	339 900 cu. yd.	
Swell, 3.2%.....		10 445 cu. yd.

Many physical analyses were made of the material in the tight section of the dam, of which those in Table 5 are representative.

Table 6 contains the results of a few percolation experiments which were made.

The temperature was not noted continuously, but was subsequently reported to have averaged approximately 70° Fahr. A correction has been applied on the basis of Hazen's formula, and the figures in Table 6 are on a basis of 50° temperature.

TABLE 5.—Physical Analyses.

Pass sieve No.	Stops on sieve No.	Percentage, by Weight.					
		A	*B*	*C*	*D*	*E*	*F*
....	4	0	0	8.45	6.40	0	0
4	10	0.52	0.16	6.70	9.38	0.56	0
10	20	1.03	0.37	11.15	16.60	3.31	0
20	40	0.76	2.34	23.59	12.26	7.03	0.50
40	60	7.68	10.66	15.52	43.45	10.16	0.58
60	80	12.21	8.16	9.65	5.75	12.69	0.84
80	100	7.70	5.83	2.41	1.73	6.25	0.58
100	200	33.70	36.99	12.60	2.93	29.14	19.00
200	....	36.48	35.49	9.93	1.50	30.86	78.50
		100.0	100.0	100.0	100.0	100.0	100.0
Taken from dam at Station.....		5 + 50	Average of 5 samples	Average of 3 samples	9 + 00	3 + 80	1 + 10
Elevation.....		2 252	2 243	2 251	2 252	2 269	2 269
Source.....		Side-hill borrow-pits.				Valley borrow-pits.	
		Pure silt.			Coarse sand found stratified in core.	Fine, black, sandy loam.	

TABLE 6.—Percolation Experiments, Conconully Dam.

Sample.	Percolation rate, in gallons per acre per day: 3 ft. of material under a 3-ft. head.	Rate observed at end of following period.
Material Deposited in Dry, Loose Condition in Tank.		
A	561 000	12 hours.
D	16 600 000	24 "
E	98 000	23 "
F	13 300	62 "
[illegible] Packed in Tank in 8-in. Layers with Iron Concrete Tamper.		
[illegible]	34 700	39 hours.
[illegible]	4 500	12 "

The high rate of p[illegible]olation in the fine silt, Sample *A*, is surprising, even though the mater[illegible]l was not compacted in the tank except in so far as it settled by subs[illegible]uent admission of water. Samples *D* and *E*

CONCONULLY DAM AND RESERVOIR COMPLETED.

showed one-third the rate of percolation when tamped, as compared with being put in loose, and if this ratio be likewise applied to Sample *A*, a rate might have been noted of nearly 190 000 gal. per day.

The sandy loam, represented by Samples *E* and *F*, may be considered water-tight. Sample *E* was taken closer to the point of discharge than Sample *F*, and averaged coarser, which explains the high rate of percolation.

Referring again to core silt, as per Sample *A*, the effect of compacting by superincumbent weight would probably reduce its rate of percolation, there being no apparent reason why this material, when well packed and with surplus water squeezed out, should not be as tight as clay.

SECTION OF MATERIAL PLACED IN DAM
AND ITS DISTRIBUTION

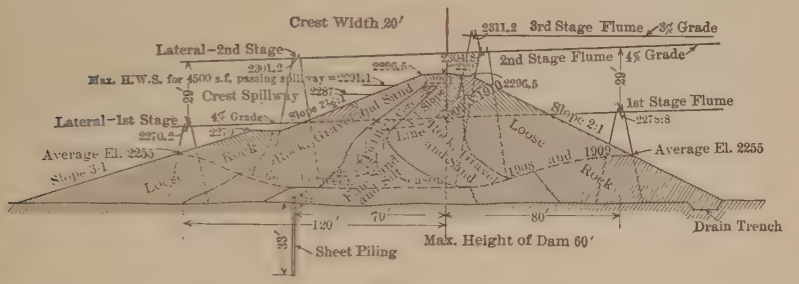


FIG. 7.

The results with Sample *D*, which was taken from a coarse sand layer which had penetrated the silt core, shows the absolute necessity of avoiding stratification, and justified the additional expense involved in the adoption of the artificial core.

The distribution of the material in the dam as built is fairly represented by the cross-section, Fig. 7, which also shows the stages of completion at the end of each season, and may with advantage be compared with the central section on Fig. 4, showing the original design.

Cost.—The total cost of the reservoir, arranged by its principal features, is shown in Table 7.

The items in Table 7 include all charges for administration, engineering, and general expenses. The cost of puddle core includes lumber for diaphragms.

The total material in the dam is 351 500 cu. yd. and the combined cost of hydraulicking and puddle core is $178 522, making the average

cost, on the basis of bank measurement, 50.8 cents per cu. yd., including lumber for core and all overhead charges, but exclusive of the cost of sloping.

TABLE 7.—Cost of Conconully Dam.

	Hydraulicking.	Main dam.	Total.	Cost per Cubic Yard, Bank Measurement. Side-hill pits, 339 900 cu. yd.	Bottom pits. 11 600 cu. yd.
Clearing reservoir site			$7 652		
Main dam:					
Creek diversion		$965			
Clearing dam site		936			
Trenches		1 862			
Sheet-piling		13 982			
Hydraulicking:					
Plant	$69 099			20.33 cents.	
Supplies	11 783			3.47 "	
Labor	74 755			21.99 "	
Total		155 637		45.79 cents.	
Puddle core		22 885			$1.97
Sloping		8 465			
Cleaning up					
Miscellaneous		1 882			
			208 219		
Outlet works			23 114		
Spillway			34 613		
Telephone			2 827		
Real estate			30 977		
General investigation of dam sites			19 028		
18 000 acre-ft., at $24.95			$324 325		

The details of the cost of hydraulicking are shown in Table 8.

The prices paid for labor were as follows:

Common labor	$2.25 to $2.50	for 8 hours.	
Pitmen	2.75 to 3.00	"	" "
Giant men	3.00	"	" "
Powder men	3.00	"	" "
Carpenters	4.00 to 4.50	"	" "
Foremen	5.00	"	" "

It will be noted that the steel lining cost 4.25 cents for material, and 1.48 cents for replacing, or a total of 5.73 cents per cu. yd. of sluiced material in the dam.

Results.—During the season of 1909 after danger from flood was past, water was allowed to rise against the dam to Elevation 2 253.3, or 17 ft. above the valley bottom. As the dam was still unfinished with the opening of spring in 1910, it was necessary to observe great

TABLE 8.—Cost of Hydraulicking, Conconully Dam.

Plant.

	Totals.	Cost, in cents per cubic yard.
Feed supply dams and flumes	$17 140	5.04
Dirt flumes and trestles, exclusive of flume lining	20 128	5.94
Steel lining	14 964	4.40
Pipes, giants, and hose	3 725	1.09
Electric light plant	1 250	0.37
Proportionate share of camp buildings	1 116	0.33
Superintendence	5 076	1.49
Administration, engineering, and general expenses	7 805	2.29
	$71 204	20.95
Less value of plant on hand	2 105	0.62
	$69 099	20.33

Supplies.

Tools	$2 150	0.63
Rubber boots and clothing	2 700	0.80
Powder and explosives	4 793	1.41
Proportionate share of camp buildings	170	0.05
Superintendence	776	0.23
Administration, engineering, and general expenses	1 194	0.36
	$11 783	3.48

Labor.

	Totals.	Cost, in cents per cubic yard, bank measurement.
Foremen	$4 274	1.25
Building road to pit	571	0.17
Clearing borrow-pit	547	0.16
Feed-supply flume tenders	7 261	2.14
Giant men	2 365	0.70
Pit men	13 232	3.89
Clearing pit of rock	8 358	2.46
Building lateral flume in pit	963	0.28
Hauling and laying pipe in pit	2 707	0.79
Dirt flume tenders	7 748	2.28
Labor, steel lining	4 975	1.46
Spreading material and puddling in dam	1 354	0.40
Carpenters on dam and flumes	3 915	1.15
Blacksmith	944	0.27
Operating light plant	767	0.24
Transportation laborers	1 308	0.38
Dismantling plant	986	0.28
Proportionate charge for camp buildings	998	0.29
Superintendence	4 543	1.34
Administration, engineering, and general expenses	6 989	2.06
	$74 755	21.99

caution in permitting storage until the work had reached spillway height elevation. Thus storage could not be commenced at an early date, and the reservoir was not filled during 1910 to an elevation higher than 2 271.5, or about 33 ft. above the valley bottom, and about 15.5 ft. below spillway height.

A small spring existed at the down-stream base of the spillway ridge, midway between the west end of the dam and the discharge end of the spillway, which had been kept under close observation, it being deemed probable that it might be supplied from ground-water from the up-stream side of the spillway ridge. This view was confirmed when the flow increased during the 1909 season from its previous steady flow of 0.115 sec-ft., to 0.21 sec-ft. as the water rose 17 ft. above the valley bottom, after which it dropped back to its original amount as the reservoir was gradually emptied.

During 1910 the water rose to 33 ft. against the dam, and the flow of the spring increased to 0.52 sec-ft., after which it dropped back to 0.10 sec-ft., or slightly less than its original flow. When the water in the reservoir reached Elevation 2 260, or 22 ft. against the dam, during the 1910 season, new springs made their appearance near the foot of the spillway, away from the dam, the water apparently coming along a line of juncture between limestone and granite. The flow gradually increased to 0.99 sec-ft., with the reservoir water at 2 271.5, and ceased after the reservoir water had dropped back to Elevation 2 260. About one-half of the above flow came from a single point.

It should be stated, in this connection, that the excavation for the spillway revealed a rather unsatisfactory rock, with seams and lines of juncture between various formations, passing clear through the ridge. The excavated material broke up quite fine, and was dumped against the reservoir slope of the ridge, but was not rolled or compacted in any mechanical way. It is not improbable that this loose dump will gradually compact, due to the action of the water, and may act to some extent as a blanket.

The springs involve no special loss of water, as the water in the reservoir will be low until the commencement of the flood season, which coincides with the commencement of the irrigation season. The water comes through entirely clear, and the leakage constitutes no element of danger to the dam or to the spillway.

The flow from Scotch Creek, at the eastern base of the gravel bar to the east of the reservoir, has remained stationary, flowing at the rate of 0.75 sec-ft. at the point where measured, a mile below the upper springs, except that it varied slightly and rose to 0.86 sec-ft. after heavy rains, and independent of the elevation of water in the reservoir.

Some water was flowing near the toe of the dam during the process of hydraulicking. Since the work stopped the flow has been measured and has fallen from 0.61 sec-ft. on July 30th, to nothing on September 10th, the reservoir in the meantime being emptied. To what extent this flow indicates leakage through or under the dam, or may be attributable to a draining out of the dam itself, is as yet uncertain.

The dam may be considered as a loose rock and gravel structure with a puddle core. Some stratification may have occurred before the artificial core was started, which would undoubtedly lead to some seepage. It is also possible that the sheet-piling may not make an entirely satisfactory connection with the side-hill rock, although the piles were driven to contact with the rock as hard as was feasible.

The construction of the dam consumed more time than was expected, due largely to a smaller carrying capacity of the hydraulicking water than had been estimated, resulting from the large percentage of rock in the pits. The aggregate contents of the pits were barely sufficient for completing the dam, and left no margin of choice in materials.

The design of the dam was approved by the Director and the Chief Engineer of the Reclamation Service, and also by Mr. A. J. Wiley, as Consulting Engineer. The designs were made and construction work was commenced under the general supervision of Mr. E. G. Hopson, as Associate Supervising Engineer, and the writer, and was completed under the supervision of Mr. C. H. Swigart. The project was in immediate charge first of C. Andersen, M. Am. Soc. C. E., and later of Mr. Ferd Bonstedt, and the work was in personal charge of Mr. Lars Bergsvik, with O. Laurgaard, Assoc. M. Am. Soc. C. E., as Engineer.

Other Earth Dams of the United States Reclamation Service.

It may be interesting to make comparison with sections of other earth dams built or being built by the Reclamation Service. Fig. 8 shows cross-sections of the following dams:

Clear Lake	Dam,	Klamath	Project,	Oregon-California.
Bumping Lake	"	Yakima	"	Washington.
Snake River	"	Minidoka	"	Idaho.
Deer Flat	"	Boise	"	Idaho.
Belle Fourche	"	Belle Fourche	"	South Dakota.
Avalon	"	Carlsbad	"	New Mexico.
Pathfinder Dike,		North Platte	"	Wyoming.

It will be noted that every cross-section shows open drainage material on the down-stream side, with the single exception of the Belle Fourche Dam, which is built entirely of clay, the nearest gravel being 5 or 6 miles distant.

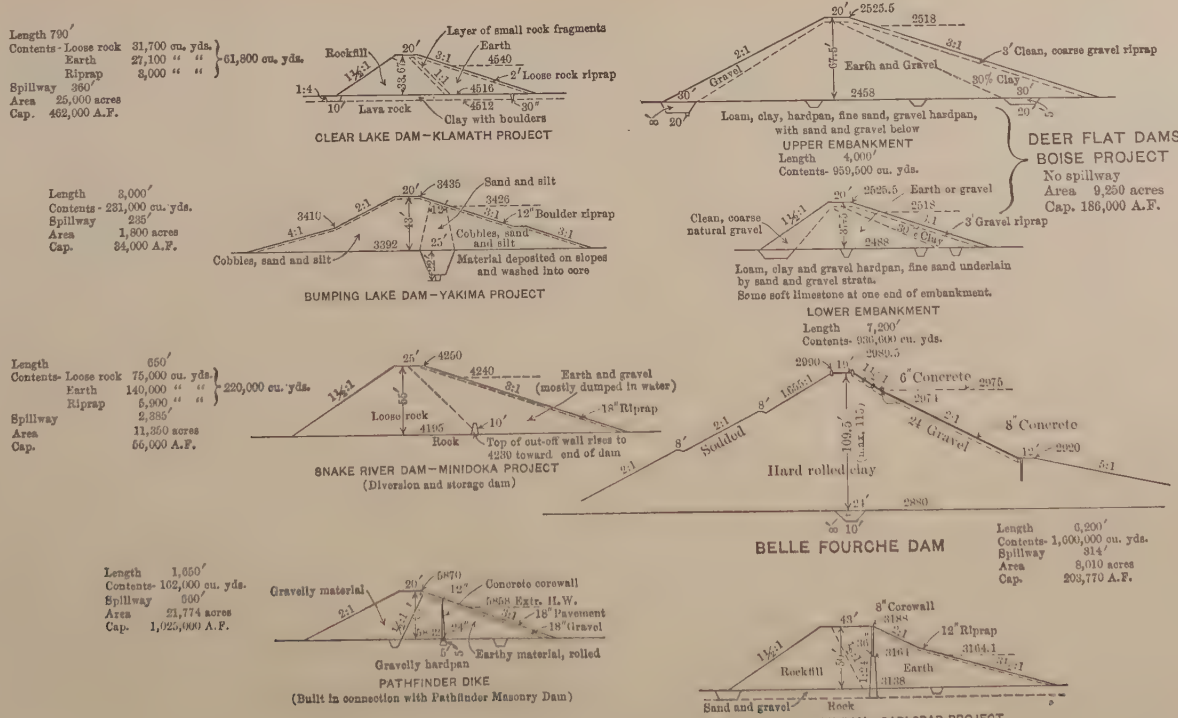
The absence of core-walls is also a general feature, barring the Pathfinder Dike and the Avalon Dam. In the former a core-wall was adopted because of the uncertainty as to the tightness of the available material, which had to be obtained from such scanty earth cover as could be found in the vicinity of the dam, or from the breaking up of the gravel hardpan, which is the only material of which large quantities were at hand.

The Avalon Dam, as a part of a private irrigation system, had twice failed. The first failure was due to over-topping. The cause of the second failure is not known. Parties having some familiarity with the conditions deny that the second failure was due to burrowing animals. The possible cause may have been the presence of a considerable quantity of soluble gypsum in the earthy material in the dam, which in canal banks has caused much trouble, and the menace of which is removed by the core-wall.

A water slope of 3:1 predominates, the Belle Fourche Dam being the most notable exception, the slope in that case being 2:1 and being faced with 8-in. concrete slabs laid on a bed of gravel.

Most of the dams mentioned above have been built by the ordinary spreading, sprinkling, and rolling method, which for briefness may be referred to as the mechanical, as compared with the hydraulic method.

In the Minidoka Dam, the earth facing of the loose rock was mechanically moved and dumped. It was dropped in water, and spreading and rolling were thereby obviated. The material was not subject to the sorting action of the water, and remained mixed as it came from the pits.



CROSS-SECTIONS OF
U.S. R.S. EARTH DAMS

FIG. 8.

In the Bumping Lake Dam, the material was moved mechanically to the slopes, mostly by dump-cars, and was washed by water-jets supplied from hose-nozzles. The large cobbles, and considerable gravel, remained in place, while the sand and silt moved into the center, the coarse sand settling on the inner slopes and the fine sand and silt in the central pond.

Hydraulic Versus Mechanical Method of Dam Construction.

The hydraulic process has been used during the last few years on numerous dams, especially in the West. It appears to have decided advantages where circumstances render it applicable, as has been fully pointed out by J. D. Schuyler,* M. Am. Soc. C. E. In observing hydraulic work of this character, the writer has become strongly impressed with certain phases of this subject, which it may be opportune to submit for consideration.

Design.—In designing a dam to be built dry, it is constructively permissible to place the water-tight element in any position in the dam where deemed best, to build it of any one of the various materials available, or any mixture of them, and to fix for it any dimensions desired.

No such freedom of choice exists with a dam to be built by hydraulicking. As to the position of the core in the dam, construction will be greatly facilitated by placing it between two dumps, and, if intended to be built of material in which clay predominates, stability may make this position practically necessary.

The core material may be obtained from a special pit, or may be washed into a pond from the slopes. In the latter case the core dimensions cannot be fixed arbitrarily, but must result from the proportionate contents of pit material, which it is difficult, and may be impossible, to ascertain definitely in advance.

If it be deemed desirable that the material in the core shall be thoroughly consolidated by drainage and pressure by the time the work is completed, another difficult problem enters into the design, as clays and silts vary greatly in regard to their retentiveness of surplus water. Little definite information is at present available on this important point.

* *Transactions*, Am. Soc. C. E., Vol. LVIII, p. 196.

Materials in Pits.—It is a rare occurrence that the material in extensive borrow-pits runs uniform. When changes are encountered during excavation the mechanical method of construction permits a ready adjustment by opening other pits, independent of distance and elevation, other than as affecting cost.

The hydraulic method, while more dependent on suitable proportions in pit material, is less elastic in adapting itself to changing and unexpected conditions, as pits must be located above a certain inclined plane, in which dirt flumes can be built, unless at the expense of general reconstruction and the possible addition of pumping.

Variation of Materials in Pits.—When the material from pits is handled dry it is constantly under easy inspection, and the disposition of the various materials can be readily controlled. If at any time the more open materials, intended for the drainage part of the dam should preponderate, that part of the dam can be made to advance for a short time ahead of the remainder.

With the hydraulic process, such adaptation is difficult, and at times not feasible, as the ponds must be maintained at slowly rising levels. At a certain stage of the work operations may be well adjusted to the momentary proportions as they are sluiced in, when an unusually coarse part of the pit may be reached, and a large excess of sand and gravel may be dumped before the necessity is realized of a more rapid change of discharge points. The sand slope will then build itself forward into the puddle pond, at first not visibly, because it is below the surface of the muddy water, and, especially when the silt-core narrows up toward the top of the dam, any stratification which may occur may easily reach through the core. Again, an excess of clay or silt may be encountered in the pit, which, if sluiced in for too great a length of time, may give rise to a layer in the dam, such as may produce local slides.

The writer has heard it argued that the silt-laden water flowing over sandy layers in the core will cause the silt to penetrate the sand, but he has never found this to be the case, unless the sand had been actually picked up by the current. Over and over again he has found clean layers of sand reaching out into the silt core without any apparent mixing, and the fact that the pond may at a certain stage be tight vertically, by no means proves that the material below is not subject to leakage horizontally. The above is in accord with the gen-

eral occurrence of pure sand strata in alluvial lake deposits, alternating with layers of silt.

Character of Core Material.—Masonry, clay, and earth mixtures have their peculiar advantages for use in cores, and each in turn may be the best to adopt in individual cases. When clay is used, it is generally the object to put it in place in a tough, thoroughly puddled, and compact condition, and to give it a position in the dam, where it will be well protected from atmospheric influences. Earth mixtures aim at obtaining tightness by the use of coarser materials, such as broken rock, or sand and gravel, mixed with sufficient clay or silt to fill the interstices, thereby securing at the same time stability and freedom from shrinkage in case of drying.

Experience with the hydraulic process is not sufficiently advanced to define with certainty the thickness of a clay core which, if hydraulicked into place, can be depended on to become thoroughly compacted in a short time by pressure and drainage. If clay is sluiced into a pond it arrives in place in a super-saturated condition, and the surplus water will only be given off slowly. If the mass be great, the interior may remain in a semi-liquid condition for a long time, endangering the stability of the entire dam.

The ultimate removal of surplus water must be accompanied by reduction of the space occupied by the clay. This implies not only downward settlement, which, if excessive, is in itself objectionable, but also horizontal contraction, causing vertical cracks, which is highly undesirable and may become dangerous.

The writer is led to believe from his experience that a tough consolidated core of large dimensions, in which the clay occupies a minimum of space, cannot be produced with the hydraulic process as usually practised. If such results have been obtained in cases which have not come to his attention, he believes that a statement of the conditions and of any special means of drainage, which may have been used, would be highly interesting.

As regards a core of mixed materials in reasonably controlled proportions, it is doubtful whether such a core can be made by the hydraulic process; for the one strongly marked feature of handling mixed materials with water is that, immediately on the formation of slopes, active segregation takes place.

Probably the nearest approach to a perfect core of great thickness

which can be hydraulicked, is one composed of fine sandy silt, such as is generally found in the arid West, having little cohesiveness, good self-drainage qualities, becoming hard and solid after a short time, and yet being, if not perfectly, at least practically water-tight.

If the core be built of whatever fine material is washed into a central pond from the slopes, there is danger of stratification, as previously stated. Opportunity for visible inspection, such as is continuously afforded with the mechanical process, exists only at infrequent intervals, when ponds are drained, and after large masses of material have been washed in. Therefore, dependence must be placed on sounding, which is very uncertain in revealing actual conditions.

In order to counteract stratification, mechanical mixing has been attempted, such as forcing down and twisting paddles, having men walk through the pond with rubber boots, etc. At best these methods are doubtful in their results. Moreover, they are likely to be entirely ineffective if stratification has proceeded beyond a certain point, for even thin layers of coarse sand offer considerable resistance to paddling.

Limiting Height.—Some of the foregoing considerations suggest that with clayey core materials, the core thickness which may be needed at the base of a high dam may be greater than can be placed with safety by the hydraulic method, by reason of uncertainty of drainage and proper consolidation. The use of such material, in connection with hydraulicking, would place a practical limit on the height to which a dam of this type can be safely built.

With the same material, placed mechanically, and without excess of water, there is no constructive limit to the feasible core thickness, and the height to which an earth dam can be safely built, as far as this consideration is concerned.

Masonry Core.—If a masonry core is adopted as part of the design, its construction interferes but little with the work, when carried on dry. In case of hydraulicking, however, the wall divides the pond into two parts, which are likely to stand at different levels and to subject the wall to undue pressures during construction. In one instance, the writer observed a large hole cut through a core-wall in a dam being built by the hydraulic process, apparently for the sole purpose of equalizing pressures.

A core-wall prevents the up-stream part of the pond from draining in a down-stream direction. Should circumstances render it

desirable to commence reservoir storage prior to completion, drainage requirements compel the maintenance of a much greater difference in level between the surface of the reservoir water and the top of the completed work than might otherwise be safe and permissible with mechanical construction; and the same holds true if the dam be given an ordinary clay or silt core.

Drainage Element.—Considering rock-fill dams as a class by themselves, there are few of the older earth dams which possess provision for drainage to the extent of having their down-stream sections consist of open material. Such provision has the disadvantage of lessening the amount of tight material which resists the flow of water through the dam, and may thus lead to increased percolation, but it has the obvious advantage of preventing the line of saturation through the dam from intersecting the down-stream slope above the lower toe. All tendency to slough is thus avoided, which may in most cases be regarded as of far more importance than a slight increase in percolation.

The hydraulic process, applied to mixed materials, is admirably adapted, through segregation on slopes, to securing such drainage element in the down-stream portion of the dam, although a similar section on the up-stream side of the dam, if it should have dimensions greater than is necessary for core protection, may well be considered objectionable, and cannot always be avoided. The above advantage becomes of little consequence where, as was the case at the Cold Springs Dam, a good drainage material is available for placing economically by the mechanical method.

Time.—The comparative amount of time required for construction must depend on local conditions, and each system may have the advantage where conditions favor it. What is almost of equal importance to the total time needed, and may be of more importance when building across streams subject to violent floods, is the ability to estimate the required time closely in advance. Where mechanical equipment is used, the factors affecting time are generally under close control. With the hydraulic method, the output is dependent on continuous water supply, percentage of solids carried, and freedom from interruption through breaking of water supply or clogging of dirt flumes. Each of these factors is likely to contain elements of serious uncertainty, to which may be attributed the apparent fact that

many hydraulic-fill dams have required far more time for their completion than had been originally estimated.

Cost.—Under favorable conditions, of sandy silt in high banks, a continuous and abundant water supply under a satisfactory pressure, and well-adjusted flume grades, the quantity of material that can be hydraulicked in a short time is remarkable, the proportion of water-carried material sometimes exceeding 25 per cent. In such cases the cost per cubic yard may be surprisingly low.

With coarse material the percentage carried by water is rapidly cut down; even with increased flume grades, the time lost by clogs may be relatively great, so that the actual performance may become small and the labor cost high.

Even with small labor cost, the plant charges of a hydraulic installation, as compared with mechanical equipment, may be so high as to throw the balance in favor of the dry method of construction.

Comparing the actual examples of construction work described in this paper, the cost per cubic yard of material in the Cold Springs Dam averaged 43 cents and in the Conconully Dam 51 cents, both based on bank measurement, while the relative plant charges, on the same basis of measurement, were approximately 12 and 20 cents, making the balance of the cost practically alike.

Such comparisons are decidedly dangerous, however, and the above should not be looked on as an argument against possible cheapness of the hydraulic process. The mere cost figures do not show in themselves the difference in conditions, and, while it is true that at Cold Springs the material had to be raised and carried farther, the costs at Conconully were greatly affected by the remoteness of the location, the necessity of stopping work over winter, and materially smaller quantities. There can be little question that the construction at Conconully under the existing conditions would have been higher in cost if any other than the hydraulic method had been used.

The difficulties, above pointed out, in estimating the required time for hydraulic construction, apply equally to cost estimates, in which respect greater certainty can probably be had when the mechanical process is adopted. This condition is reflected by the usual inability of obtaining unit contract bids on hydraulic work, although this is undoubtedly due in part to the lack of contractors who have even a limited experience in this line of construction.

Conclusion.—The foregoing comparison has been made in some detail to indicate what are believed to be the principal factors, which should receive attention, when the application of the hydraulic method to the construction of large earth dams is under consideration.

It appears to the writer that the hydraulic process has important advantages in special cases, and also that it has decided limitations, not at present well defined. Its potential economical possibilities may at times be utilized to their full extent, and it may, under exceptional conditions, accomplish results not otherwise readily obtainable.

The mechanical process has the important advantage in the construction of high earth dams, designed to contain great masses of clay or material in which clay largely predominates, of permitting immediate consolidation and avoiding the dangers from instability and possible future excessive settlement and shrinkage, which may result from initial surplus water, such as would be necessarily introduced by the application of the hydraulic process.

In cases of doubt, the hydraulic and mechanical methods being equally applicable and showing little difference in cost, it is believed that the mechanical method deserves the preference, by reason of greater certainty of results as to quality, time, and cost.

DISCUSSION

JAMES DIX SCHUYLER, M. AM. SOC. C. E. (by letter).—This paper is a timely and interesting account of the construction of two earth dams, under somewhat unusual and rather difficult conditions, by distinctly different methods, and the facts presented add most acceptably to the sum of engineering experience in such constructions. It is to be hoped that all Engineers of the U. S. Reclamation Service who have had charge of building dams will confer a similar favor on the Profession by publishing in detail the results achieved on works of such magnitude, carried out under the wide range of conditions which the Western field presents. Mr. Schuyler.

The author compares the building of earth dams by the old-fashioned mechanical means of delivering, spreading, sprinkling, and rolling the earth, with the more recently developed method, which involves the use of water as the principal agent for transporting, sorting, and consolidating the mass; and apparently draws conclusions which are not altogether favorable to the hydraulic method. The writer cannot subscribe to these conclusions, as he does not feel that they can be fairly drawn, at least from the experience in the cases cited. The cost per cubic yard of material in the Cold Springs Dam, placed by mechanical means, is given at 43 cents, which is sufficiently below the average of such work elsewhere to indicate that it was managed with economy, skill, and care. The average cost per cubic yard of material placed by the hydraulic method in the Conconully Dam is stated to have been 50.8 cents, which is so much higher than such construction should ordinarily cost, that it seems to indicate exceptionally difficult conditions, or lack of experience, or both. The author is fair enough to admit that

"There can be little question that the construction at Conconully, under the existing conditions, would have been higher in cost if any other than the hydraulic method had been used."

This admission covers one of the most important of the advantages presented by the hydraulic system of dam building, as it frequently offers the opportunity of accomplishing results, such as were attained at Conconully, where other methods would have failed or have been prohibitive in cost. This being true, as appears to have been demonstrated at Conconully Dam, the merits of the hydraulic method are certain to become more conspicuous as experience develops the best standards of working.

Conditions at Conconully were evidently extremely unfavorable for economical or efficient operation of the system, and it cannot be taken as a criterion from which to judge the merits of the hydraulic-fill process of dam building. A dam, the total volume of which is only

Mr. Schuyler.

351 500 cu. yd., but which required a plant costing $71 200, with a scrap value of only $2 105 when the work was finished, apparently presents extremely difficult conditions or shows inexperienced management. It is rather remarkable that more than 20% of the cost of hydraulicking is represented by the items of "Superintendence, Administration, Engineering, and General Expenses"—items which are usually amply covered by the ordinary 10% of engineers' estimates for "Superintendence and Engineering."

It is evident that the attempt to place the impervious core material farther up stream than the center of the dam, where it has been located in all other hydraulic-fill dams of which the writer has knowledge, the sheet-piling cut-off having been located 70 ft. above the center line, resulting in the necessity for inclining the plane of the core, and confining it between inclined board forms, added very largely to the cost and difficulty of building the dam, especially as the core material for its upper part was handled by scrapers from the valley borrow-pits. From the description of all the materials available, and from the plans and photographs submitted, the writer believes that the work could have been simplified and cheapened by the following modifications:

First.—By providing a larger water supply at the outset, instead of working a whole year with an inadequate volume, to move, on the flat gradients chosen for the flumes, the angular rock which the borrow-pits yielded.

Second.—By building the flumes for delivery of material on minimum grades of 6%, instead of 4%; 8% would have been better.

Third.—By building the bulk of the dam as a simple embankment, without attempting to segregate the material further than necessary to prevent waste by providing for the settlement of the fines in a pond between levees, but filling this pond periodically and systematically with rock, sand, gravel, and clay, the run of the pit.

Fourth.—By depositing a heavy, impervious blanket, of selected fine material from the valley borrow-pits, on the up-stream face of the main embankment previously built as suggested, this blanket to serve as a water-tight cover for the dam, already well consolidated by the water and the ramming impact of material dropping from the flumes. It would preferably be considerably thicker at the bottom than at the top, as it should have a flatter slope on the water side than the rock. This water-tight facing, as a substitute for the central core, apparently could have been handled more cheaply by a hydraulic elevator, as there was sufficient available head in the flume to serve the purpose, for at least two-thirds the entire height.

The suggested plan of working would have made the dam practically a rock-fill structure with earth facing, similar to the Milner and

Minidoka Dams, on Snake River, Idaho, the Zuñi Dam, in New Mexico, and the Clear Lake Dam of the Klamath Project. Mr. Schuyler.

The author has fairly stated the principal factors which require careful consideration when the application of the hydraulic method to earth-dam construction is proposed. The most ardent advocates of the method have never claimed that it could be made applicable to every condition, that it was automatic in its action, or that it required any less intelligence and watchful care, skill, and experience to produce the best results. What is specially true of the hydraulic method, however, is that where it is applicable, and skillfully used, it will frequently give cheaper results and better dams, and, therefore, is worthy of consideration at all times, and in every case. Even though the material may be excavated by steam shovel and delivered by cars to the dam, sluicing may often be used to advantage for segregating the fine from the coarse to secure water-tight material for the core, as was done at the Arrowhead Dam, in California, and the Willow Creek Dam, in Oregon. Undoubtedly there are many cases where sluicing can be used to better advantage than the mechanical process, and at much less cost, and not infrequently it can be used to build a high-class dam where it would have been absolutely impracticable to build a safe structure by any other method. In the writer's opinion the great Gatun Dam, by far the largest in the world, in which the bulk of the material is being handled satisfactorily by hydraulic dredges, could not have been built so well, so cheaply, or so rapidly, by the ordinary dry mechanical means. Some years ago a dam was built on the site of James Peak, in Colorado, at an altitude of more than 10 000 ft., where the material had to be sluiced out from between boulders so large and numerous as to have made it impossible, or impracticable, to select and segregate what was required by any mechanical means.

One of the special features of interest in the paper is the description of the foundation which "could not be made water-tight," and the use of sheet-piling, driven 33 ft. into the alluvial soil of the valley, simply to "retard percolation" and to discourage seepage by increasing the length of its path from the reservoir to the exit below the dam. The amount of settlement of the dam into its unstable alluvial foundation, due to a compacting of the fine loamy sand, to the extent of 3 or 4 ft., is rather extraordinary and unusual, and further illustrates the difficulties of the situation. The future history of this structure will be instructive and interesting, and if it endures the test of time and satisfactorily performs its functions, it will establish a valuable precedent and extend the range of feasibility in dam construction.

The absence of core-walls from five of the seven earth dams built by the U. S. Reclamation Service, cited in the paper, other than the

Mr. Schuyler. two described, is evidence of an inclination on the part of the engineers in charge to depart from the hidebound Eastern American practice of using core-walls of masonry or concrete, which are so frequently an element of weakness rather than of service in promoting water-tightness.

The writer feels greatly indebted to the author for the clear presentation of his experiences and opinions, but hopes that his future experience with the hydraulic process may lead to additional illumination of a subject which is still somewhat obscure in the minds of the majority.

Mr. Henny. D. C. HENNY, M. AM. SOC. C. E. (by letter).—The constantly accumulating experience in hydraulic dam construction renders any expression of opinion by its foremost advocate of great value. The discussion contributed by Mr. Schuyler, therefore, places the writer under obligation. It, moreover, enables him to supply such further explanation of the design and construction details of the Conconully Dam as appears to be necessary in order to avoid misunderstanding.

Certain suggestions were made in the discussion regarding the manner in which the work might have been cheapened and simplified. To an open-minded engineer, suggestions of this general nature will occur after the work is completed and its difficulties and cost have become definitely known. Nevertheless, the special points raised in the discussion are not applicable to the actual case in hand.

It is axiomatic, in hydraulic dam construction, that an abundant water supply, great working pressure at the nozzles, and steep grades for dirt flumes are desirable, and each of these features received rather exhaustive consideration before the hydraulic method of construction for the Conconully Dam was decided on and the hydraulic plant was designed.

The only stream available for diversion for construction purposes was not used the whole year, with an inadequate water supply, as was assumed in the discussion, but was utilized in connection with small storage reservoirs at the head during short working seasons, as stated on page 65. The time of actual sluicing was cut down, in order to permit these reservoirs to fill between shifts, the number of sluicing hours being shown by Table 4.

The increase in flume capacity, after the first operating season, from 17 to 26 sec-ft. was rendered advisable by the excess of angular rock found in the pit material over what had been estimated on the basis of elaborate test-pit investigations. It was obtained only at the sacrifice of further operating time.

The grade of the dirt flumes was made as steep as circumstances would permit. The adopted grade of 4% left barely sufficient material and working pressure between the upper stretch of the dirt flume and

Mr. Henny.

the gravity feed canal, and reached the location of the dam at an elevation hardly sufficient to permit its completion by hydraulicking, the final work requiring a partial flattening of this grade to 3%, as stated on page 67. It would have been impracticable to raise the feed canal materially and thus permit a steepening of the grade of the dirt flume, as this would have necessitated feed-water diversion at a higher elevation on the stream, with a consequent decrease of available flow and excessive increase in flume length and cost.

Undoubtedly an 8% grade for the dirt flume, as suggested, would have expedited and cheapened the work, had it been practicable; but its adoption would have left no material to transport, and no working pressure at the nozzles, or would have caused the dirt flume to intersect the valley bottom before reaching the dam. The foregoing emphasizes the rigid limits which the hydraulic process is apt to impose on construction methods in practice.

It was also suggested in the discussion that the work might have been cheapened by a change in dam design. A hydraulicked dam was proposed, not tight in itself, but to be made tight by an earth facing to be placed by a hydraulic elevator. Such general design had been considered, and had been rejected because of its higher estimated cost, largely due to the difficulty of obtaining suitable earth.

The suggested design seems particularly unsuited to the hydraulic process, which in this case had the special advantages of providing by far the greatest amount of core material by sluicing, and of furnishing the necessary rip-rap in place for water slope protection.

It is difficult to understand how the proposed change, sacrificing the most obvious advantages of the hydraulic process, could have cheapened the work.

The inclined position of the puddle core has not materially added to its cost. As long as it could be formed from the silt in the hydraulic pit material, it merely required keeping the down-stream dump piles higher than those up stream. The wooden diaphragms, subsequently used, were made necessary to insure against stratification, and not by the inclined position of the core.

This much regarding the suggestions for the simplifying and cheapening of the work at Conconully.

In the discussion it was rightly surmised from the description that the conditions at Conconully for securing low unit cost were unusually adverse, by reason of remoteness of location, scarcity and high cost of labor, short working season, in addition to the nature of the material to be handled and its relatively small quantity.

It may be stated here that the 8-hour law, applicable to Government work, added 25% to all labor cost, railroad contractors in the

Mr. Henny.

same region paying no more for 10 hours of labor than the Government paid for 8 hours.

The item of cost representing "Superintendence, Engineering, General Expenses, and Administration," was commented on as unusually high. It should be remembered, however, that it included all charges of a general nature, such, for instance, as legal expenses, and the cost of the Washington office. It is realized that such general charges are seldom included in published cost statements, and it was done in this instance to conform to U. S. Reclamation Service bookkeeping. The item is given separately, and, for comparative purposes, can be readily reduced to suit circumstances, if desired. Slow progress and the necessity of extending the work over nearly four seasons have tended also to increase this item.

Mr. Schuyler remarks that the average cost per cubic yard of hydraulicking at Conconully was much higher than such work should ordinarily cost. It appears to the writer that there can be no question of ordinary cost in work of this character, considering the variableness of the conditions.

The economy in special instances cannot be properly judged by what the unit cost may have been elsewhere. Moreover, in attempting to make such comparison, one is confronted with an almost total lack of reliable information. Most work of this character has been done for private corporations, which do not generally favor the publication of their cost data. Statements, as far as published, are apt to be partial, and to refer to some short period of work, often selected for the reason that it established a favorable record. They rarely include expenses of a general nature, and sometimes omit the plant cost. Such statements, therefore, must be used with great caution.

As an instance, reports on the cost of the hydraulic work for strengthening the down-stream slope of the San Leandro Dam, in California, range from 5 to 20 cents per cu. yd.

The Crane Valley Dam, in California, as built before its recent reconstruction, cost, according to the best information available, five times the published amount of the estimate.

In regard to the Terrace Dam, in Southern Colorado, the only cost data published gave the labor cost of the best average 30 days at 6.5 cents per cu. yd., and the total plant cost at $34 500. This would show a final low unit cost. Some three years ago the writer visited this dam, and found that the work had been stopped, only a small portion being completed. The reason for stoppage was said to have been the excess of the actual over the estimated cost. This work has been resumed from time to time, but, as far as the writer is aware, has not yet been finished.

Referring again to a mere superficial comparison of the unit cost

of hydraulic construction at Conconully with such cost elsewhere, the writer was much interested in ascertaining the unit cost of the noted Necaxa Dam, at the time of his visit some 2½ years ago, shortly after the slide. The yardage is about six times that of the Conconully Dam, to which the slide has added one-third. The cost of common labor was low. He was somewhat surprised to be informed by the engineers in charge that the plant cost per cubic yard was practically the same as at Conconully (or twice as much, expressed in Mexican currency). Final and accurate information on these points should be very welcome to the Profession. Mr. Henny.

It was stated in the discussion that conclusions had been drawn, not altogether favorable to the hydraulic process. This is correct, in the sense that the intention has been to show its limitations as well as its advantages, to caution against an excess of enthusiasm on the part of some of its advocates, and to point out the factors which in the writer's judgment should receive attention. It is a source of satisfaction to him that the fairness in the presentation of these factors has been acknowledged in the discussion.

He has not made his meaning clear, however, if his conclusions are interpreted as a criticism of the hydraulic process. He stated specifically that he believed it to have important advantages and potential economical possibilities in special cases, and that it might, under exceptional conditions, accomplish results not otherwise readily obtainable. Therefore, he fully agrees that this process should be carefully considered in each case of dam construction.

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TRANSACTIONS

Paper No. 1206

PIONEER RAILWAY DEVELOPMENT IN THE UNITED STATES.

By W. D. Taylor, M. Am. Soc. C. E.

With Discussion by Messrs. Marshall R. Pugh, R. L. Morrison, William J. Boucher, George T. Hammond, and Fred. J. Wood.

The First Chartered Railway.—Railway development proper began in England. In 1801—in the very beginning of the 19th century, the century which was to witness the most wonderful material achievements of the human race—the first chartered line of rails was laid for a short horse-car line from Wandsworth to Croydon, in the suburbs of London.* Lines of similar character were constructed almost every year thereafter.

An Important Discovery.—In 1813 a discovery was made which led to the adoption of steam as a propelling power, instead of horses or stationary engines. It was discovered that cars could be propelled along a line of rails by the adhesion of a round smooth wheel to the head of a smooth rail. When Blackett made the experiment, however, he did not revolve his wheel by steam power.

"Six men were placed upon a frame, which was fitted up with a windlass attached by gearing to the several wheels. When the men worked the windlass, the adhesion was found sufficient to enable them to propel the machine without slipping."†

* Hadley's "Railroad Transportation," p. 9.
† Brown's "History of the First Locomotives in America." New York, 1871, p. 45.

The First Railroader.—George Stephenson, who was born on June 9th, 1781, in a colliery village on the north bank of the Tyne, became the forerunner and prototype of the modern resourceful and self-reliant railroader. In his childhood, wagons loaded with coal were pulled by horses over the tramway which passed his home. In his boyhood he worked in and about the coal mines, and became familiar with steam engines as they were used at that time, for pumping and hoisting in mines. It is said that at seventeen he was engineer of a pumping engine, with his own father as fireman under him. Stephenson constructed his first locomotive in 1814, and his second during the next year. He made many improvements in his successive locomotives. In 1822 the tramways leading to and about the Hatton Colliery, in Durham, had been converted into a locomotive railroad, and Stephenson was Chief Engineer. Concerning the day when the use of the steam locomotive was inaugurated, a writer says:

"Five of Mr. Stephenson's engines were upon the road that day, each engine drawing after it seventeen wagons loaded, averaging sixty-four tons, at the rate of four miles an hour."*

Railways or tramways propelled by horses had now been in operation many years in the transportation of coal, ores, and quarry products. As early as 1800 the use of horse tramways for purposes of general traffic between cities had been advocated by a Mr. Thomas, who brought the subject to the attention of the Literary and Philosophical Society of Newcastle. In 1820 Thomas Gray advocated the use of locomotives instead of horses as a means of propulsion on tramways, and published pamphlets and articles on the subject. He also advocated the building of the Liverpool and Manchester Railroad and many similar constructions. While Gray was getting public sentiment ready for the use of locomotives in general traffic, Stephenson was perfecting his locomotives, and learning to build the roads that were needed. The opening of the tramways of the Hatton Colliery as a steam-operated railroad, on November 18th, 1822, was attended by great crowds who came to witness the performance of the engines.

The Stockton and Darlington Railway.—At about this time Stephenson became Chief Engineer of the Stockton and Darlington Railway, a road about to be built for the purpose of tramming coal,

* Brown's "History of the First Locomotives in America," p. 55.

The surveys had already been made when Stephenson was engaged, at £300 a year. His influence was not sufficient at this time to persuade the managers of this enterprise to lay out the road so as to have it operated by steam power, although they had been in person to see his locomotives working at Killingworth, where they had then been used successfully for ten years; but he succeeded in having certain changes made, and three locomotives ordered, so that he could at least compete against horses and stationary engines for the form of power to be used. The opening of the Stockton and Darlington line on September 27th, 1825, is thus described by Smiles:

"At the foot of the incline a locomotive was in readiness to receive them, Mr. Stephenson himself driving the engine. The train consisted of six wagons loaded with coals and flour; after these was the passenger coach, filled with the directors and their friends, and then twenty-one wagons fitted up with temporary seats for passengers; and lastly came six wagon-loads of coals, making in all a train of thirty-eight vehicles. The local chronicler of the day went almost out of breath in describing the extraordinary event: 'The signal being given,' he says, 'the engine started off with this immense train of carriages; and such was its velocity, that in some parts the speed was frequently 12 miles an hour!' By the time the train reached Stockton there were about 600 persons in the train or hanging on to the wagons, which must have gone at a safe and steady pace of from four to six miles an hour from Darlington."*

The First Locomotive Works.—A little more than a year before this, Stephenson had persuaded the chief projector of the Stockton and Darlington Railroad to become his partner in the organization of a factory at Newcastle for the construction of locomotives.

The Traffic on the Stockton and Darlington Railway.—The working of the Stockton and Darlington Railway was a surprise to its most sanguine projectors. The line was constructed without any view whatever to passenger traffic, and with little expectation of doing anything more than to supply the local demand for coal at the stations along the route; but the existence and operation of the road soon did what has been done in thousands of instances since then—it created a traffic which was non-existent before. The first passenger coach was run on this road; it had been built at Stephenson's factory at Newcastle. It was propelled by horses, and its operation at first was leased to private parties. On this road it was first demonstrated that a

* Smiles' "Lives of the Engineers," Vol. V, 1874, p. 137 (or Vol. III, 1868, p. 168).

railway cannot be operated like a highway, with the patrons of the road owning the vehicles and regulating their movements. The gauge of the tracks of this road was fixed so as to accommodate the greatest number of trucks then used on the highways, most of which were 4 ft. 8½ in. from in to in of the wheels.

The Liverpool and Manchester Line.—The success of the Stockton and Darlington road, and of the locomotives that Stephenson built and put upon it, led the way to the construction of the more famous Liverpool and Manchester Railway, and to the use of steam power thereon. It is well to note that the Stockton and Darlington Railway was built because a canal was not feasible or was too expensive; but the Liverpool and Manchester Railway was projected because the canals between the two towns could not take care of the existing traffic. The Public Declaration in regard to this railroad, drawn up in Liverpool in 1824, and signed by 150 or more of the principal merchants, explained that they considered "the present establishment for the transport of goods inadequate, and that a new line of conveyance has become absolutely necessary to conduct the increasing trade of the country with speed, certainty and economy."*

Motive Power for the Liverpool and Manchester Line.—Even in 1829, when the construction of the Liverpool and Manchester Railway was nearing completion, the form of motive power to be used was a mooted question. Several impractical methods were proposed, but the directors were halting between horse-power and stationary engines. The locomotive, on account of its supposed dangers and inconvenience to the public, had few adherents. It was claimed that the gases exhaled from it were so poisonous that birds flying across the track in its wake fell down dead, and that all vegetation would be killed for great distances on each side of the road.

In this year was conceded the Newcastle and Carlisle Railway Act, which definitely excluded locomotives, and fixed upon horse-power as the traction to be used. Even while this question was being fomented, Stephenson had some of his locomotives at work hauling at the excavations for the construction of the road. The contemplation of the immense traffic that would come to the road finally excluded the consideration of horse-power, and, in the spring of 1829, a committee of two eminent engineers was appointed to report on the best form

* Smiles' "Lives of the Engineers: The Locomotive." John Murray, London, 1874, p. 154.

of power to be used on the new road. These engineers concurred in the opinion that the first cost or interest charge of the plant to operate the road would be greater if stationary engines were used, while the working expenses would be greater if locomotives were used. On the whole, stationary engines would be cheaper in operation. The cost per ton-mile of traffic would be 0.28d if locomotives were used, and 0.21d with stationary engines. In the report of Mr. J. Walker, one of the two engineers referred to, the conclusions arrived at are thus summarized:

"Our opinion is that if it be resolved to make the Liverpool and Manchester Railway complete at once * * * the stationary reciprocating system is the best; but that if the power was to be proportioned to the demand then locomotives would be preferable."

In their plan of working the road, the 32 miles were to be cut up into 21 sections, with fixed engines to propel the trains. The final offer of £500 as a prize by the directors for a locomotive which should best fulfill certain conditions, and the winning of the prize by Stephenson's *Rocket* in October, 1829, is now familiar history.

Passenger Traffic.—It is interesting to note that, even with the example of the Stockton and Darlington Railway before them—considerable passenger traffic having been developed where none existed before—the management of the Liverpool and Manchester line made little or no provision in their outlay for passenger traffic.* The road was constructed entirely with a view of taking care of heavy traffic, such as coal, cotton, timber, and merchandise, whereas the receipts from passenger traffic in this densely settled locality soon outgrew those from freight. The massiveness of the construction in this early line was somewhat startling, and the cost would be considered prohibitive, even at this day, on all but the most important lines in America. The road was more than four years in active construction, and was double-tracked from the beginning. There was a tunnel 1½ miles long at Liverpool, and a 2-mile cutting, for the most part in solid rock, and 80 ft. deep at some points. The cost of the construction and equipment amounted to more than $130 000 per mile. The rail was light, weighing 35 lb. per yd., and, for the most part, was of

* In a description by an early passenger over the road occurs the following: "The first class of train is the most fashionable, but the second or third are the most amusing. I travelled one day from Liverpool to Manchester in the lumber train. Many of the carriages were occupied by the swinish multitude and others by a multitude of swine." ("Railroads: Their Origin and Problems," by Charles Francis Adams, Jr., G. P. Putnam's Sons, New York, 1878, p. 35.)

forged iron. Each rail was 15 ft. long, and was supported every 3 ft. on stone blocks. The construction of the Manchester and Liverpool line and the successful use of Stephenson's locomotives thereon were such wonderful achievements, and became so well known, that all opposition to the new mode of locomotion and of moving traffic was broken down, and the English people entered upon the first era of railway construction.

Railway Beginnings in America.—Other countries were watching these interesting developments in England, and none with more vigilance than the United States.

In December, 1824, there was organized the "Pennsylvania Society for the Promotion of Internal Improvements in the Commonwealth." Early in 1825 this Society sent William Strickland, an architect, to Europe, to collect information in regard to canals, roads, railways, steam engines, and the industrial arts. The result of Strickland's researches was scant, as far as concerned information on railways; but, from the elaborately prepared instructions to him as to the objects to be obtained by his visit, some insight can be gained as to the attitude of mind of the more intelligent Americans in regard to the development of transportation matters.

"'It is not a knowledge of abstract principles, nor an indefinite and general account of their application to the great works of Europe, we desire to possess through your labors. * * * What we earnestly wish to obtain, is the means of executing all those works in the best manner, and with the greatest economy and certainty; and for these purposes you will procure and exhibit in your reports, all that will enable those who shall undertake the formation of canals, railways, and roads, * * * to perform the work. * * * To use a term which is familiar to you as an architect, we desire to obtain working plans of the best constructed canals, and their locks and inclined planes; of railways, and all means of using them to advantage; of roads and of the mode of their formation and preservation; * * *

"'*Canals and Railways* present the most important of all subjects for your attention. Upon every matter connected with both you will be expected to be well informed; and if you shall have to decide between them you must be able to furnish the facts and circumstances by which the decision shall be produced. Much excitement prevails in this state upon the question whether railways are superior to canals, * * *.'"*

* "Development of Transportation Systems in the United States." J. L. Ringwalt, Philadelphia, 1888, p. 70.

The United Kingdom was covered by a network of good highways and canals. Railroads were necessary only as a means of further development and because traffic had overgrown the other means of transportation. In 1830 the United States had few canals and few highways. There was great dearth of the means of intercommunication. In England the introduction of the railway was on a grand scale befitting the establishment of a means of intercommunication which was to be the beginning of another era of civilization.* The constructions were massive and the outlay great. The opening of the Liverpool and Manchester line was an event which attracted the presence of persons famed in history.

In America everything connected with the early railways was meager and poor. Had it been necessary to begin on the magnificent scale on which the Liverpool line was laid out, the development of America must have waited; but railways made a beginning in America almost as soon as in England. There is some reason to believe that the first railroad ever constructed with a view of using steam as a motive power was built in the United States.

The First Tramway in America.—In 1826 a tramway some 3 miles long was built at Quincy, Mass., for conveying stone for the Bunker Hill Monument. It was operated as a tramway until 1871, using only horses as a motive power. Its cost was $34 000. Instead of cross-ties, it had granite blocks laid transverse to the line every 8 ft. The rails were of pine, 12 in. deep. The top of the pine rail was covered with a strip of oak, and this in turn with flat strips of iron. The gauge was 5 ft., and the grade was descending from the quarry to the Neponset River. On it two horses could draw a load of 40 tons. This is frequently mentioned as the first American railroad; but it was not a railroad at all until 1871, when it was bought up by the Old Colony Railroad. The old structure was demolished and a modern road was built on the right of way.†

The Mauch Chunk Tram.—The next American tramway seems to have been built in 1827 from Mauch Chunk, Pa., to the Lehigh River, a distance of 9 miles. It was built to convey coal from the mines to the river boats. The descent was very great, and the cars went down by

* G. S. Morison's "The New Epoch as Developed by the Manufacture of Power," Boston, 1903.

† Adams' "Railroads: Their Origin and Problems," p. 38.

gravity when loaded and were drawn up to the mines by mules when empty, though, according to some accounts, the cars were pulled up hill by a stationary engine.* The construction of the track was similar to that of the Quincy tram, except that, for cross-ties, wooden sleepers were used.

The First American Railways.—The South Carolina Railroad was opened for traffic on January 15th, 1830. It was early projected to unite Charleston Harbor and the mouth of the Oriskany. The road was constructed with a view of using the locomotive. A queer-looking machine, named the *Best Friend,* was built at the West Point Foundry Works, in New York, in the summer of 1830 for use as a locomotive on this road. It was certainly no infringement on any patent of Stephenson's. The boiler was vertical, and of the porter-bottle shape. The furnace at the bottom was surrounded by water, and some additional heating surface was supplied by projections or "teats" running inside from the sides and top. As originally constructed, it had double crank axles, wooden wheels, and iron tires. This engine arrived in Charleston in October, 1830, and made several trial trips before the celebration of the anniversary of the opening of the road, in January, 1831. It was said to have reached speeds of from 16 to 21 miles an hour with 40 or 50 passengers in 4 or 5 cars, and from 30 to 35 miles an hour without cars. One hundred and fifty or more of the stockholders and their guests, with bands playing, with a field piece and a detachment of United States troops, paraded over the road and "great hilarity and good humor prevailed throughout the day." Not long after this celebration—in the following June—the *Best Friend* came to grief. The negro fireman, annoyed by the popping off of the safety valve, held it down and both the fireman and the *Best Friend* were incapacitated for further service.† In 1834 this road was 135 miles

* Poor's "Manual," 1869-70, p. xxii.

† In May, 1831, an accident occurred in the operation of the Charleston and Hamburg which necessitated an explanation from the Chief Engineer to the President of the Company:

"CHARLESTON, May 14, 1831.

To ELIAS HORRY, Esq., President.—

"Sir:—I hasten to communicate the causes which produced the accident of yesterday afternoon. It originated in the wild derangement of the tongue, which guides the wheel through the turnout, by some ill-disposed person, and was rendered injurious to the car by the imprudent speed allowed by those who had the management of the engine. the tongue having been nailed to its proper position, but was made loose by removing the fastening, and was probably shaken from its place by the speed with which the engine and one car had preceded the one injured. Directions have been given to pass the turnout at moderate speed, and the attention of the person in charge to be constantly kept on the road in advance of the engine.

"Respectfully your obedient servant,
HORATIO ALLEN."

in length, running from Charleston to Hamburg, on the Savannah River, opposite Augusta, Ga. The grades did not exceed 30 ft. per mile. The "plan of the road is unusually straight and the curves have large radii." The track was composed of flat iron bars attached to longitudinal 6 by 10-in. stringers. Timber pile trestles were used in lieu of embankments to such an extent that "the railway resembles a continuous and prolonged bridge."* The exposed part of the timber was "protected by a coating of heated tar and oil." The cost of the line was less than $13 000 per mile, including equipment. It was sold before 1840 to another company for $17 500 per mile. During November, 1839, it earned $65 000, which was "nearly 50 per cent. more than was taken in any previous month."

In 1829 three locomotives were secured from Stephenson's works at Newcastle, for the Delaware and Hudson Canal and Railroad Company, and on that company's road, in August, 1829, and probably on the 8th day of the month, at Honesdale, Pa., the first locomotive driving wheel was turned by steam power in America. These three engines were failures, however, in two ways: They did not have Stephenson's last improvement of the multitubular boiler, and they were too heavy for use on the tracks on which they were placed.

The Baltimore and Ohio.—The pioneer American road for purposes of general traffic was the Baltimore and Ohio, which was chartered on February 28th, 1827, though construction did not begin until 1828. Even then it was slow in building, and only 13 miles were ready for traffic in 1830. However, 135 miles had been built in 1835. The road was organized to try to regain the Western traffic, which had been deflected from the Cumberland road by the opening of the Erie Canal. Eminent French engineers had been employed to survey a canal across the Alleghanies to connect with the waters of the Western rivers. The expense was entirely beyond the resources of the people. The railway was organized simply because a canal could not be had. At the outset it was expected that horses would be the motive power. A car to be propelled by sails was put on the road, and worked very well—when the wind blew strong and from the right quarter.

It was on this road, on August 28th, 1830, that a trial of speed between Peter Cooper's locomotive, *Tom Thumb,* with Peter Cooper

* "A Description of the Canals and Rail Roads of the United States," by H. S. Tanner, New York, 1840.

himself as engineer, and a horse, each drawing equal loads, resulted in a victory for the animal and not for the machine. The *Tom Thumb* weighed only 1 ton, and the diameter of its cylinder was 3½ in. Stephenson's success in his great contest was due to two great improvements in *The Rocket:* one was the waste steam blast through the smokestack, to increase the draft, and the other was the multitubular boiler, to increase the heating surface. Cooper had built the *Tom Thumb* with the latter improvement, but not the former. Not being able to secure the usual tubes, Cooper had used gun barrels in his boiler. The steaming power of the *Tom Thumb* was dependent upon a special device to increase the draft, which device failed on the occasion of the race mentioned. Peter Cooper, however, built the *Tom Thumb* only as a working model, to demonstrate his belief in the adaptability of the locomotive to the sharp curves in the tracks of the railways of this country which were made necessary by economy. That the locomotive could be adapted to as sharp curves as were used on American roads was denied by many of the most eminent European engineers. The European roads were straight, or had very gentle curves; but Cooper ran the *Tom Thumb,* without any appreciable decrease of speed, around curves of less than 300 ft. radius.* The performance of this engine, however, on the tracks of the Baltimore and Ohio, resulted in a much greater benefit than in making this demonstration.†

The construction of this road, from Baltimore to Ellicott's Mills, was very expensive, one excavation near Baltimore being 70 ft. deep and costing $122 000. Between Baltimore and the Potomac at the

* John H. B. Latrobe's Lecture before the Maryland Institute, March 23d, 1868.

† Ross Winans, who had witnessed the tests of the *Rocket* in England, also rode behind Cooper's engine on the Baltimore and Ohio. From a long letter to the President of the Company, dated August 28th, 1830, the following abstracts are taken:

"Mr. Cooper's engine has to-day moved a gross weight of four and a half tons from the depot to Ellicott's Mills and back in the space of two hours and ten minutes, which, as you are aware, the distance being twenty-six miles, gives an average speed of twelve miles to the hour. As the engine returned with its load to the same point whence it started, the acclivities and declivities of the road were, of course, balanced;

* * * * * * * * *

"To-day's experiments must, I think, establish, beyond a doubt, the practicability of using locomotive steam-power on the Baltimore and Ohio Railroad for the conveyance of passengers and goods at such speed and with such safety (when compared with other modes) as will be perfectly satisfactory to all parties concerned, and with such economy as must be highly flattering to the interests of the company. It has been doubted by many whether the unavoidable numerous short curves on the line of your road and the inclined planes would not render the use of locomotive-power impracticable; but the velocity with which we have been propelled to-day by steam-power round some of the shortest curves (to wit, from fifteen to eighteen miles per hour) without the slightest appearance of danger, and with very little, if any, increased resistance, as there was no appreciable falling off in the rate of speed, and the slight diminution in speed in passing up the inclined planes, some of which were nearly twenty feet to the mile, must, I think, put an end to such doubts, and at once show the capability of the Baltimore and Ohio Railroad to do much more than was at first anticipated or promised by its projectors and supporters." (Brown's "History of the First Locomotives in America," pp. 118 and 119.)

Point of Rocks, 68 miles out, there were thirty-one stone viaducts. In 1840 the capital of the road was $5 000 000, and the line had been built to Harper's Ferry, 80.5 miles from Baltimore. It was also located to the Ohio River at Wheeling, but this construction was not completed until 1852, at which time it was the longest railroad in the world. The Washington Branch of this road was opened for traffic on August 25th, 1835. The first trains were spoken of as "a brigade of cars." The *Baltimore American* of May 24th, 1830, said: "A brigade of cars will run three times a day each way from Baltimore to Ellicott's Mills—passage 25 cents."*

The Early Passenger Cars.—The early passenger cars on the Baltimore and Ohio Railroad were horse cars "like a market-car on railroad-wheels. Then came cars resembling the old-fashioned stage-coach, with the same springs and leather braces. * * * For a long time the cars were gaudily painted. * * *"† Ross Winans, who also was the real inventor of the "friction wheel" and of the "outside bearing," on which principle the world's rolling stock is now constructed, introduced on this road, prior to 1833, the first eight-wheeled car ever built for passenger service. It was named the *Columbus,* and had four-wheeled trucks at each end.

Steel Springs.—Another great improvement, made in the early days of the Baltimore and Ohio, was the introduction of steel springs under both locomotives and cars, to relieve the jolting. This experiment was first tried on the locomotive *York,* in September, 1832, and was found to work so well that springs were added to the cars as well. At the great opening of the Liverpool and Manchester, the distinguished visitors must have had a very rough ride, unless Stephenson had put his road in uncommonly good line and surface beforehand; for the early trains went pounding and thundering over the track at a fearful rate, before the roughness was relieved by steel springs. An early director of the Baltimore and Ohio proposed to relieve the jolting by

* The description of the track of the Baltimore and Ohio road given in Tanner's "Canals and Railroads," p. 150, is instructive:

"The superstructure is various: it consists of stone sills, stone blocks and wooden sleepers, on different parts of the line. Forty miles of single track are composed of granite sills, 8 inches thick, 15 wide, and of various lengths: these are laid in trenches, filled with broken stone. Six miles of single track are composed of stone blocks and wooden string-pieces, 6 inches square. The blocks are 4 feet apart, from centre to centre. The line hence to the Potomac, rests on wooden sleepers, four feet apart and imbedded in broken stone. The sleepers are hewn out in the centre to make room for the horse-path. The iron bars are 15 feet in length (pierced by 11 oblong holes) 2½ inches wide, ¾ inch thick, and bevelled at the ends."

† Brown's "History of the First Locomotives in America," p. 101.

putting a thin sheet of lead between the iron rail of the track and the supporting stone stringers, to act as a cushion. The Baltimore and Ohio was the pioneer railway line of America in other ways than that of priority. The early results attained by this road are thus described:

"In the construction of the road from Baltimore to the Point of Rocks, every mode hitherto suggested by science or experience had been tested, and thus the work must be regarded as having the honor of solving most of the problems which presented themselves in this early period of railroads in this country. The granite, and the iron rail; the wood and iron, on stone blocks; the wood and iron on wooden sleepers, supported by broken stone; the same supported by longitudinal ground-sills in place of broken stones; the log-rail, formed of trunks of trees, worked to a surface on one side to receive the iron, and supported by wooden sleepers; and the wrought-iron rails of the English mode—had all been laid down, and as early as 1832 formed different portions of the work. * * * Their road and workshops have been a lecture-room to thousands who are now practising and improving upon their hard-earned experience."*

Early New York Roads.—The initial trip of the locomotive in New York State was over the Mohawk and Hudson Railroad from Albany to Schenectady on August 9th, 1831. The sketch of this first train in New York State, drawn by the *DeWitt Clinton,* and made up of stage-coach cars placed upon trucks, is now familiar to every schoolboy. The *Albany Argus,* of September 26th, 1831, gives an interesting account of a trip of certain officials over this road. Concerning the dining in Schenectady after a trip up from Albany, the account says:

"Among the toasts offered was one which has been verified to the letter, viz.: 'The Buffalo Railroad—may we soon breakfast in Utica, dine in Rochester, and sup with our friends on Lake Erie!'"†

If one were to ride on the fastest trains on the New York Central to-day, he would find an opportunity to eat those three meals inside of four hours. Tanner's description of the track of the Mohawk and Hudson, written about 1840, reads as follows:

"The cross sleepers are of wood, 7 inches in diameter, and 8 feet in length. The iron plate is a bar, ⅝ by 2½ inches, with the upper curves rounded to 1¾ inches width. The stone blocks are laid three feet apart from centre to centre, on broken stone, and on these the timber rails

* Brown's "History of the First Locomotives in America," p. 140.
† *Ibid*, p. 186.

are placed. The width between the rails is 4 feet 9 inches. The company was chartered in 1826, with a capital of $600 000, or about $38 000 per mile; and the work was commenced in 1830; double track completed in 1833."*

The line from Schenectady to Utica was not completed until about 1836, and continuous connection by rail from New York to Buffalo was not obtained before 1842. The Erie Canal had then been opened 17 years.

Early Pennsylvania Roads.—By the end of 1835 Pennsylvania was well in the lead of any other State in railway construction, 265 miles being open for traffic. The Columbia Railroad (now a part of the Pennsylvania) from Philadelphia to the Susquehanna River, was opened in 1834. By the end of 1837 a chain of canals and railroads joined New York, Philadelphia, Baltimore, Wilmington, and Pittsburg, thus bringing the interior of the country along the Ohio and Mississippi Rivers into closer commercial relation with the Atlantic seaboard.

Early Massachusetts Roads.—The chartering of railroads began in Massachusetts in 1830, and by 1835 three lines leading out of Boston, to Providence, Lowell, and Worcester, were constructed. These were all opened for traffic within four weeks of each other, in the summer of 1835. It is interesting to note that in America many of the early engineers learned to build cheap roads. Oftentimes, when English methods were adopted, failure resulted. Charles Francis Adams, Jr., former President of the Union Pacific, says:

"It is, indeed, matter of curious observation that almost uniformly those early railroad builders made grave blunders, whenever they tried to do their work peculiarly well. * * * The Lowell road, for instance, was too well built in many respects. On a portion of its track the stone blocks, into which the oaken plugs to spike the rails to were inserted, were laid on a foundation of continuous, parallel, dry, stone walls running in trenches under each line of rails, and from two and a half to four feet deep and a foot and a half wide, * * * and to-day the sides of the road-bed are liberally ornamented with those useless stone sleepers, the eternal life of which was once looked forward to with confident pride."†

The Worcester line was saved from such blunders by the good judgment of its constructing engineer, who, although he had been sent to

* Tanner's "Canals and Railroads of the United States," p. 77.

† Adams' "Railroads: Their Origin and Problems," pp. 74 and 75.

England to study English methods of construction, saw that the objection to the use of wood in England, on account of scarcity and cost, did not prevail in the United States. When the Erie Canal was completed, and the commerce of the West flowed into New York, Boston was eager for a similar connection to the West. As early as 1826, the year after the Erie Canal had opened, the Legislature of Massachusetts had appointed a commission to survey a route for a canal from tide-water at Boston to the Connecticut, and thence to the Hudson. A canal running across the Berkshire Hills was not a particularly feasible engineering undertaking; but the canal was projected on paper, and it was to have a 4-mile tunnel on the site now occupied by the Hoosac Railway Tunnel.* The project lay dormant, however.

In 1829 the *Boston Daily Advertiser* gave full accounts of the performance of the *Rocket* and of the construction of the English railway lines. It also gave full accounts of the spectacular opening of the Liverpool and Manchester line. The charters of the three roads previously mentioned were granted by the Legislature, as a result of the success of the English roads and locomotives.

The Western Railroad, as it was called, was begun in 1834, from Worcester west through Springfield to the western edge of the State, where it connected with another road which joined it to the Hudson. Through many financial troubles its construction was continued until its completion in 1841, when Boston was at last connected with the stream of Western traffic.

The First Era of Railway Construction.—To no other country could the advent of the railway in 1830 have meant what it did to the United States. The country's inhabitants were scattered along lines of water communication, along the coasts of the Gulf and of the Atlantic, along the shores of the Great Lakes, and along the navigable rivers and the canals. President Hadley says:

"The system of internal water-routes is a thing of the present in more senses than one. When no longer used, its influence is still felt, because it has laid down the lines of the country's development."†

The riches of the great interior country, of the splendid forests of timber, of the fertile interior valleys, of the bounding prairies, and of

* Adams' "Railroads: Their Origin and Problems," p. 53.

† Arthur Twining Hadley's "Railroad Transportation." G. P. Putnam's Sons, 1886, p. 28.

the coal, iron, copper, gold, and silver fields, were largely known and perhaps wholly anticipated; but they were all unavailing for lack of highways of commerce. In Colonial times it took a week to go from Boston to New York by stage, a distance which the New Haven road hopes soon to cover by a 4-hour train. Between these points there was a tri-weekly mail service. President Hadley says, of the conditions of the country at a period immediately succeeding the Revolution:

"Long-distance freight movement was absolutely impossible. The charge for hauling a cord of wood twenty miles was three dollars. For hauling a barrel of flour one hundred and fifty miles it was five dollars. Either of these charges was sufficient to double the price of the article and set a practical limit to its conveyance. Salt, which cost one cent a pound at the shore, would sometimes cost six cents a pound three hundred miles inland, the difference representing the bare cost of transportation. It was on these cheap articles of common use that the charge bore most heavily. It forced every community to live within itself."*

The use of turnpike roads, which were built to provide for through traffic, began in America in 1790. The system was pretty well developed in certain parts of New York, Pennsylvania, and Southern New England. Except in Kentucky and Tennessee, the turnpike system, south of the Ohio and the Potomac, never amounted to much. These roads were maintained by a system of tolls. National aid for turnpike roads began to be agitated in 1808. The plan of national development of the turnpike proposed three radiating lines from Washington, leading to Detroit, St. Louis, and New Orleans. It included a line of road along the coast from Maine to Georgia, and a line of roads and canals connecting the Mississippi and its tributaries with the cities of the Atlantic seaboard. Most of the agitation and legislation on the subject came to nothing. Almost the sole result of national aid to turnpikes was the Cumberland road, connecting Washington, Wheeling, Columbus, and Vandalia, on which the Government spent somewhat more than $4 000 000. The mania for turnpikes was succeeded by a mania for canals. The receipts from the Erie Canal nearly doubled in the period from 1826 to 1833. The canal rates were reduced about 40% in 1834, and the traffic of the Canal was increased so much under this encouragement that the receipts continued to increase; they exceeded the expenditures by $942 000 in 1833, and by $1 125 000 in

* Hadley's "Railroad Transportation," p. 25.

1836. Thus the Canal lowered rates, but increased commerce, and made money. It brought commerce in such a stream to New York City that that city was forging ahead of all its competitors. At Utica, Syracuse, and Rochester, large cities were springing up along the course of the Canal. The increase of population in New York State from 1820 to 1840 was 1 056 000, while, in the same period, the increase was only 215 000 in Massachusetts, and 63 000 in Maryland. With such results before them, it was small wonder that Massachusetts contemplated building a canal across the Berkshire Hills; and Baltimore, across the Alleghanies. Canals, however, could not be constructed so as to accommodate the traffic of all parts of the country. While they were very successful in Pennsylvania and New York, and in a large part of the territory between the Lakes and the Ohio, those of Virginia and Indiana were not prosperous.

Two great internal lines of water transportation have exerted and retained a marked influence on the development of the United States, and only two: one running east and west through the Great Lakes and the Erie Canal, and one running north and south along the Mississippi and its tributaries. These routes continue to exercise a potent influence over commerce, even to this day, though at present the canals and rivers are all but deserted by the commerce of the country. The construction of canals was terminated somewhat abruptly in 1837 by the financial crisis, and, by the time the reaction set in and normal conditions returned, public opinion had decided that these systems of public works had not been beneficial, and that the interior commerce of the country was not to be carried on by water. By 1840 there had been constructed 2 800 miles of railway, and the United States may then be said to have been fairly launched on a period of railway construction, and to have been committed to the policy of developing the resources of the country by means of railways.

Conditions at the End of the First Decade of Railway Construction.—All railway construction had been in response to a well-defined need. The early railroads were built to connect points between which traffic already existed and was carried on by less convenient means. The country had not yet had enough experience with the results of improved means of intercommunication to justify railroad extension into new country into which the railway should bring the population from which its traffic was to be derived. The railway was something

new, and its rules of operation and development were unknown. These could not be learned except in the school of experience. The only modes of interior transportation heretofore had been highways and canals. Ideas born of the operation of both turnpikes and canals were dominant in the development of the early railways. This was apparent in the use made of the railways, in the forms of construction adopted, and in the legislation affecting them. It required years of experience to prove that the railway was neither a turnpike nor a canal.

Even by the construction of roads to connect one locality with the next, however, something like systems of development were forming. Many of those engaged in railway building failed to see the trend of this development; but, that the country was looking for such a method is evident from the system on which the proposed national turnpikes were planned. Even at this early date, four separate chartered railway companies between Philadelphia and Baltimore had combined to form one.

By some a principal line of communication was thought to be forming between the crest of the Alleghanies and the Atlantic Coast, extending all the way from Portsmouth, N. H., to Pensacola, Fla. This was to become the channel along which the main stream of American traffic should flow. Other lines of communication were to intersect this at angles more or less acute, cutting through to the cities of the coast and going back into the interior of the country along the most available lines of least resistance. Among these secondary traffic routes were those from New York and Boston westward along the Hudson and Mohawk Valleys and the shores of the Lakes, from Philadelphia westward along the Susquehanna and the Ohio, from Baltimore westward along the Potomac and the Ohio, and from Richmond westward along the James, the Kanawha, and the Ohio. This great future traffic route would not cross the mountains, though it would cross the streams. Great canals could not cross the mountains, and it had not yet been shown that great railways could. Peter Cooper had demonstrated at the outset that the track need not be straight, but, before 1835, no one had shown that it need not be level—or nearly so. On these early railway lines the great ascents were overcome by inclined planes operated by stationary engines. The Stockton and Darlington Railway had such planes, and a stationary engine hauled the cars up

on one side and lowered them on the other. The Liverpool and Manchester line avoided the planes, except at three points, by cutting through the elevations on a more expensive plan than American lines could adopt. The steepest grade used, except on the planes, was 4 ft. per mile. Consequently, in 1840, the railways in America were full of planes. The celebrated trip of the *DeWitt Clinton* took place from the "top of the plane at Albany" to "the head of the inclined plane at Schenectady."* A part of Tanner's description of the Baltimore and Ohio reads as follows:†

"First inclined plane, 41 miles from Baltimore, ascends 80 $\frac{375}{1000}$ feet in a distance of 2 150 feet. The second plane ascends 99 $\frac{805}{1000}$ feet in 3 000 feet. The summit, at Parr's Spring Ridge, is 813¼ feet above mid-tide. The line then descends by an inclined plane, 3 200 feet long, descending 159.63 feet, and another, 1 900 feet long, and descends 81.350 feet. All the planes are straight."

The most noted example of the use of these planes, however, was on the Portage Railroad, over the Alleghanies between Hollidaysburg and Johnstown, on the through-traffic route between Philadelphia and Pittsburg, over practically the present route of the Pennsylvania Railroad, which route was completed and opened for traffic in 1834. The distance from Hollidaysburg west to the summit was 10 miles, and 1 200 ft. vertical of the 1 400 ft. difference in elevation were overcome by five inclined planes interspersed with more level portions of road. The combined length of these planes was 13 500 ft. From the summit west to Johnstown the distance was 26.5 miles, and the difference of elevation was 1 170 ft. On the descent there were also five planes having a total length of 9 670 ft., on which the descent was 800 ft. The gradients on the lightest of these planes was 7 in 100, and on the heaviest, 9.8 in 100.‡

There was an inclined plane at the quarry on the Quincy tramway. During the construction of the Boston and Providence Railroad a party of gentlemen was taking a pleasure ride up this plane when the pulley broke and the car ran backward down the plane, throwing them out and killing one of them. Mr. Adams says that this incident illustrated "the value of fatal accidents as a dynamic force in railroad

* Brown's "History of the First Locomotives in America," pp. 176 and 180.
† Tanner's "Canals and Rail Roads," p. 149.
‡ Sipes' "The Pennsylvania Railroad." Philadelphia, 1875.

development."* This accident brought the inclined plane into disfavor, and the Boston and Providence excluded them, at great cost. Although other roads deplored their use, they did not know how to avoid them. In 1840 Charles L. Schlatter, Principal State Engineer of Pennsylvania, made his report to the Canal Commissioners of Pennsylvania on a survey for a continuous railway from Harrisburg to Pittsburg. In this report it was demonstrated for the first time that the Alleghany Mountains could be crossed without the use of inclined planes.

The value of Western connections was not under-estimated. In 1840 Tanner wrote:

"The immense effects which a connected system of improvements between the East and West, must produce in securing the trade of the latter to the Atlantic states and in cementing that bond of union upon which the safety of our institutions depends, would furnish, upon the proper occasion, a subject of interesting and profitable inquiry."

The early practical railroad men wasted no time in trying to develop systems of road parallel to the ocean front. At Richmond, Baltimore, Philadelphia, New York, Boston, and Albany, attention was concentrated on the construction of lines toward the West—toward the great interior of the country.

The cars and locomotives of 1830 were very light. Freight cars carried from 2 to 3 tons of freight, and weighed empty from 1 to 1.5 tons. In 1831, on the Baltimore and Ohio Railroad, it required a train of eight cars to transport 200 bbl. of flour, the loaded train weighing 28 tons. The chief engineer of the Baltimore and Ohio said in 1832:

"In the year 1828 the power of the locomotive engine was no more than sufficient to propel itself up an ascent of 1 in 96 at the rate of 10 miles an hour, without dragging any load after it. In the course of two years after, however, such were the improvements made in this engine that it could draw up that ascent a train of cars weighing, with their freight, 17 tons, at 10 miles per hour. At the same time, it could draw on a level, at the same speed, 53$\frac{1}{2}$ tons; at 15 miles per hour, 30 tons, and at 20 miles per hour, 15 tons."†

In 1831 the advertisement of the Baltimore and Ohio for American engines read:

"The engine, when in operation, must not exceed three and one-half

* Adams' "Railroads: Their Origin and Problems."

† Ringwalt's "Transportation Systems in the United States," p. 99.

tons' weight, and must, on a level road, be capable of drawing, day by day, fifteen tons, inclusive of the weight of the wagons."*

The first English locomotive, imported in 1826, weighed only 7 tons, and yet it had to be discarded because the track could not carry it. The strap-iron rail and wooden stringer track retarded the use of heavier locomotives, but trains rapidly increased in weight. The first annual report of the Erie road, in September, 1835, said:*

"The board of directors now have the gratification of announcing to the stockholders the following result, to wit: That loads of sixty tons gross (or, deducting the weight of the cars, forty tons net), may be drawn in a single train from the Hudson river to lake Erie, and at an average speed of from twelve to fourteen miles to the hour; * * *."

It was said in 1840 that an engine weighing 12 tons pulled a train of 470 tons gross weight behind the tender over a level or descending portion of the Reading road 54 miles in 5½ hours. On the return trip it pulled 182 tons behind the tender up a grade of 18.4 ft. per mile. In 1840 the Baldwin Locomotive Works had already constructed 137 locomotives for 39 different companies in 17 different States.

One of the most troublesome features in early railroading was the rate of wear of the imperfect iron rails used. The distinguished civil engineer, Charles Ellet, Jr., writing in the *Mechanics Magazine* (New York) in April, 1844, said that eight or ten of the railroads of the country had worn out the common ½-in. flat bar rail by carrying 150 000 net tons of freight over their lines. The Camden and Amboy wore out its 40-lb. "edge rail" with 400 000 net tons of traffic. In 9 years the rails of the Liverpool and Manchester line had to be entirely replaced four times. From 1825 to 1842 the net traffic on the Stockton and Darlington, which had been carried in light cars at speeds of 6 miles an hour, summed up 6 500 000 tons. Besides a great amount of patching, the rail of the track had then been entirely renewed six times. The London and Birmingham started in constructing its line with 50-lb. iron rail, and it was worn out before the road was completed. Ellet, in the article referred to, calculated that the cost of rail wear alone per ton-mile of net freight on the Reading road was 4.75 mills, a figure greater than that at which coarse freight can now be carried at a profit by probably all the larger railways of the country.

* Ringwalt's "Transportation Systems in the United States," p. 99.

The telegraph service was not available for railway operation until about 1850. With no telegraph or telephone connection, with no steam whistle on the engine, no air-whistle or bell-cord signals, and—above all—with no uniform code of operating rules, early railway operation was crude in the extreme. Seemingly, the only practical method affording an approach to regular operation consisted in rigid adherence to time cards; and this was impossible on account of frequent derailments, the crude tracks in use, and the frequent breaking down of engines and cars. On busy roads locomotives were kept ready with steam up at convenient points to go to the relief of any belated train. Usually, the only warning that could be had of any trouble was the fact that a train did not arrive when expected. When the relief engine started out to meet the delayed train there was no means of telling whether the delayed train was just around the first curve or 20 miles or more away. Therefore, on a single-track road, it was necessary to flag around every curve. Headlights on night service engines with "powerful reflectors" were introduced on the Boston and Worcester in August, 1840.

In the very early days the engine driver collected passenger fares, and the brakeman handled the heavy baggage and the local freight that went by the same train. There was no conductor. Later, important trains were put in more responsible charge of an officer called the captain of the train. There was a deal of responsibility and authority to the old post of train captain or conductor, which no longer pertains to it. His position was somewhat similar to that of the captain of a sailing vessel, who is in responsible charge of the movements of the vessel, and of its passengers and its cargo, throughout the journey of uncertain duration. Some of the early train captains were elected by ballot by the board of directors of the company.

The report, in January, 1840, of the directors of the Camden and Amboy Railroad and of the Delaware and Raritan Canal showed some interesting results. The length of the journey from New York to Philadelphia had been decreased "from 11 to 20 hours to from 6 to 7 hours." The results of the operation of canal and road were as follows:

Expenses in construction	$6 064 953
Share capital	2 900 000

The remaining capital was borrowed at 6 per cent. For seven years there had been an annual increase in the profits of about 20%, and if this increase continued for seven years more the property would then be earning 28% dividends. The operating account for the seven years was:

Year.	Gross receipts.	Operating expenses.	Net receipts.
1833	$468 142	$287 092	$181 050
1834	546 993	313 262	233 731
1835	679 464	317 492	361 972
1836	770 621	363 345	407 276
1837	731 995	359 510	372 485
1838	754 990	355 249	399 741
1839	685 330	258 043	427 287

The report, further, showed that the railway had been very profitable, while the canal had barely paid expenses. The canal had cost $2 830 000, and had yielded a net profit of $96 500 in six years. The railway had cost $3 235 000, and in the same six years had yielded a net profit of $2 203 000.*

In 1840 Northern railroads generally, except certain coal roads, earned more from passengers than from freight; the Southern roads earned more from freight. Flour, coal, tobacco, and cotton were the principal commodities carried. Freight rates were so high that many articles now carried by rail could not bear the cost of movement. Live stock was driven on foot. Whisky was carried largely on the common roads. In the early operation of the Liverpool and Manchester the freight charge per ton-mile was between 8 and 9 cents. On the Baltimore and Ohio it was 6 cents, and it was about the same on the State road from Philadelphia to Columbia. In 1840 the flour rate from Pittsburg to Philadelphia was $1.55 per bbl., which was a little less than 4 cents per ton-mile. In 1838 the Mohawk and Hudson, in competition with the Erie Canal, carried flour at 4 cents and light goods at 6 cents per ton-mile. The Central Railroad, operated by the State of Michigan, charged 12½ cents per ton-mile on flour in 1838. The Petersburg Railroad, in Virginia, was prohibited in its charter from charging more than 12½ cents per ton-mile. In 1840 this same road was permitted by the Legislature to charge passengers 8 cents a mile. The Central Railroad and Banking Company, of Georgia, was prohibited in its charter from charging passengers more than 5 cents a mile. In 1837 passenger rates per mile were as follows on the lines

* Ringwalt's "Development of Transportation Systems in the United States," p. 77.

named: Baltimore and Ohio, 3 cents; Baltimore and Washington, 6 cents; Portsmouth and Roanoke, 6 cents; Boston and Providence, 5 cents; Boston and Lowell, 3½ cents; Mohawk and Hudson, 5 cents.*

A number of the States, such as Pennsylvania, Georgia, and Michigan, entered into the business of railroad construction and operation. State aid to railways in various forms was quite prevalent. In two of the Southern States, Georgia and South Carolina, State aid was successful. It was unprofitable in most other States. In these early days it was difficult to secure enough capital to build any great length of railway or canal unless the credit of the State stood behind the enterprise. Before 1842 Massachusetts had extended assistance to railways amounting to $5 500 000. In 1840 New York had helped railways to about the same extent. In 1839, $18 000 000 had been expended for railway construction in Pennsylvania, nearly $6 000 000 of which had been expended by the State on its own railways. New Jersey stood firm against all State aid. The Camden and Amboy, and the Delaware and Raritan Canal were constructed by a private company, which contributed about $3 000 000 in stock, and as much more was borrowed in England without the credit of the State. Pennsylvania found her canals burdensome and her railways unprofitable, though no great burden. In May, 1864, Commodore R. F. Stockton, who represented the New Jersey companies, wrote as follows:

"The experience of several states in the management of railroads and canals has proved that politicians could not do as well for the state with the public works as they could do for themselves. * * * In 1830 the people of New Jersey * * * had perceived the demoralization and deterioration of virtue, which already began to be flagrant in those states in which lavish expenditures for internal improvements, and vast state patronage, incidental to state management of public works, had taken place. Particularly they had marked the headlong and reckless precipitancy with which corrupt demagogues had plunged the noble state of Pennsylvania into the vortex of enormous debt."†

About 1840, Charles Ellet, Jr., wrote concerning the financial situation of American railways, as follows:

"'Some few have thus far sustained themselves, and distributed considerable dividends. The receipts of some others are sufficient to

* Authority for the amount of these rates and fares may be found in Ringwalt's "Development of Transportation Systems in the United States," pp. 106 to 111.

† Ringwalt's "Development of Transportation Systems in the United States," pp. 79 and 80.

keep them in repair, and pay the interest on the loans incurred for their construction; but of the balance, having an aggregate length of some two thousand miles, the capitals may be regarded as positively sunk, and many of the companies as insolvent. * * * They make costly roads, build expensive superstructures, rear extravagant edifices to contain their cars and engines, run heavy locomotives, and use carriages almost as capacious as dwelling houses, to carry as many passengers as could, without much inconvenience, be drawn in a hand-cart. If railroads do not sustain themselves, it is not because they are railroads, but because great roads have been constructed where little ones only are required. * * * The power contrived to turn a grist mill would make but small dividends if applied to turn a churn.' "*

The Country's Commerce, as Affected by Railway Development up to 1840.—Up to 1840 the railroads, though of great convenience and value to certain localities, had exerted little influence on the country's commerce. There was little interstate movement of the heavy commodities of modern traffic. There were but two transportation routes in operation connecting the Central West with the Atlantic seaboard cities: the Erie Canal and the State route through Pennsylvania, composed of part canal and part railroad. This second route, up to 1840 and for long years afterward, had little effect in deflecting traffic from the Erie Canal and New York City to Philadelphia. The annual east-bound traffic over the Portage Railroad during its period of operation, from 1834 to 1854, was said to be not more than 20 000 tons. From 1837 to 1854 the east-bound tonnage on the Erie Canal was never less than 612 000, and in 1853 it was 2 506 000. Thus far in railway development there had been realized little besides good grounds for high hopes for the future. The first continuous line of railway built with a view of reaching and controlling the streams of internal traffic was the Western Railroad of Massachusetts.

The Settlement of the Middle West.—As far as the West had been peopled, up to 1840, the year when railway traffic may be said to have begun to affect the development of the country, though the effect was feeble at first, it had been done by people traveling by raft or flatboat, or by saddle and pack-horse. The settlers carried to their new homes axes, rifles, bullets, and salt, and little else. Once over the Alleghanies, the streams flowed westward, and it was easy to go down with the

* Ringwalt's "Development of Transportation Systems in the United States," p. 81.

current, but hard to move up against it. It was easy to get the produce of the West to New Orleans, but difficult to get even the person of the hunter or settler back over the Alleghanies. Therefore, many went and did not return. From 1790 to 1840 the population was moving steadily westward down the streams. In 1790 the center of population was east of Baltimore, in Chesapeake Bay. By 1840 it had traveled westward 221 miles and was across the Alleghanies. The Louisiana Purchase had placed the Mississippi River from end to end in United States territory. On the plains of the West the settler easily produced more than he consumed, and his surplus products went down stream with the river currents. The West was breaking away from the East.

The Westward Development of the Railroads.—The last link in the lines of railway between Albany and Buffalo was completed in 1842; but, at that time, no one thought of the railway as competing for the traffic of the canal. The lines from the Ohio River and the Lakes eastward never entered into competition with the canals until other railways were constructed from their western termini still farther west into the great interior. The traffic collected on the Lakes naturally clung to water routes all the way to the seaboard, while it was found that traffic originating along the railway lines in the interior tended to cling to the rail routes until it reached its final destination. By 1848 a line was constructed through from Lake Erie *via* Springfield to the Ohio at Cincinnati. The Cleveland and Pittsburg was put in operation in 1852, making another junction of Lake Erie and the Ohio, and the line from Buffalo to Toledo along the Lake in 1853. The Michigan Central and the Michigan Southern were opened in 1852. The Rock Island reached the Mississippi River from Chicago in 1854. In 1851 the Hudson River Railroad was opened, and in the same year the Erie was opened from the Hudson to Lake Erie. The Pennsylvania was opened through to the Ohio at Pittsburg in 1852, and the Baltimore and Ohio reached the Ohio in 1852.

In order to prevent the railway from competing with the Erie Canal, all freight over the New York Central was subjected to canal tolls, soon after the line was opened through; but, when the Erie road was opened, in 1851, on which no such charge could be placed, the Central was relieved of the canal tolls, and then the competition between the canal and the railway began in earnest.

"From 1853 to 1859 there was a fight for supremacy between canal and railroad. For twelve years more there was a contest for profits. Then it became a question whether the canal could pay expenses of maintenance; a question which was finally decided in the negative."*

The through east-bound traffic on both the Erie and the Central in 1852 footed up only 46 601 tons, while the canal carried east a through traffic twenty-five times as great; but in the next year it was only fifteen times as great. In 1854 the ton-mileage on the canal decreased by 31 731 000, while the ton-mileage on the Ogdensburg, Central, and Erie roads increased by 55 736 000. In the same year the revenues of the canal fell off $431 152, and those of the three railways named increased $1 520 688.†

Succeeding the great business depression of 1837, all public improvements in the United States were at a low ebb. The discovery of gold in California in 1848 instilled new hope and confidence, and the material development of the whole country comprising the United States may be said to date from this event; but the completion of the Erie road from the Hudson to Lake Erie and the consequent relief of the Central from canal tolls in 1851 may together be said to mark the real beginning of modern railway transportation methods and practice in the United States. Up to this date railway traffic had exercised small influence on the commerce of the country, except between local communities. The bulk of the railway business had been due to passengers. Even in 1851 the total gross income from passengers and from freight was about equal, $19 000 000 from the one and $20 000 000 from the other. Some idea of the magnitude of the change begun then may be obtained by considering the fact that in 1867 the freight business was 70% of the whole, a proportion which has since been fairly well maintained. The freight proportion in 1870 was 73%; in 1880, 76%; in 1892, 69%; in 1909, 75 per cent.

The successful competition of the railways with the canal was distressing to the adherents of canal traffic, but it meant the salvation and expansion of the great interior country. The West was now ready to return to the East. After being separated by water traffic, the two sections were to be reunited by rail traffic. The Pennsylvania Railroad was extended west from Pittsburg to Chicago by the completion of the Ft. Wayne road in 1858.

* Hadley's "Railroad Transportation," p. 30.

† Poor's "Manual," 1881, p. xxv.

From 1850 to 1860 the railway mileage of the country increased 240 per cent. In the same decade the center of population made the largest stride it has ever made; from 1790 to 1900 it traveled westward 519 miles, or an average of 47.2 miles per decade, but from 1850 to 1860 it traveled westward 81 miles.

The Beginning of Railway Consolidation.—Separate short roads were now being combined under one management. In 1853 the New York Central was formed by combining what had been eleven companies. By 1858 this company had gained control of five other lines. Later there followed the union with the Hudson River road, the Harlem, the Lake Shore, the Canada Southern, the Michigan Central, and the New York, Chicago, and St. Louis, making up a system of 4 000 miles. The action of this line was typical. As long as traffic was local, each community could organize and run its own road to suit itself, but, when it was shown that the railways were to be the chief

FIG. 1.

means of transporting through traffic, economy and satisfactory service both required uniformity in all conditions affecting traffic movement, and this was most easily secured by combining separate managements into one.

Railroad Construction Between Chicago and the Missouri River.—The reaching of the Mississippi by the Rock Island in 1854 was an important step. The produce of the country west of that stream, as well as that along the Illinois and the Wisconsin Rivers, heretofore had gone down stream. The Rock Island was closely followed by the Galena and Chicago Union (Northwestern), by the Chicago and Alton in 1855, and by the Chicago, Burlington, and Quincy in 1856. Lake Michigan and the Mississippi were united farther north by the Milwaukee and Prairie du Chien in 1857, and in 1858 by the Milwaukee and La Crosse (now the St. Paul), and in the next year the Hannibal and St. Joseph reached the Missouri River.

The Illinois Central was conceived and constructed to develop the Prairie State of Illinois. There were 705 miles in the original road. It was built in the form of a Y, standing north and south. Dubuque, Iowa, was at the apex of the western prong, Chicago at the other, while Cairo, at the junction of the Ohio and the Mississippi, was at the end of the tail. The first train was run in 1855, and the 705 miles were completed in 1856. The road paralleled watercourses, but was constructed with a view of bringing traffic to Chicago for shipment through the Lakes and the Erie Canal. By the time the road was made ready for traffic, however, the influence of the east and west roads was so strong in swerving traffic from the Lakes and the Erie Canal to their lines that the Illinois Central was brought face to face with a failing business in 1857, when the panic caused railway securities all over the country to fall in ruins. The remedy for the troubles of this line was eventually found in the construction of a north and south trans-continental line.

Trans-Missouri Development.—The settler and the hunter, ever pressing westward, found their progress a different matter after crossing the Mississippi and the Missouri. The current was now against them. It was 1826 before the first steamboat reached Lake Michigan, 1828 before it reached the Great Falls in the Missouri, and 1832 before it reached Chicago. But the West had found rude ways of getting up stream before the steamboat came. The fur trade in the North, the merchant trade in the far Southwest, and the freighting traffic to the mining regions of the West, each developed individual systems of transportation which were alike interesting and instructive. The natural routes to the Great West now lay along and up the Rio Grande, the Red, the Arkansas, the Platte, the Yellowstone, and the Upper Missouri; but the prices that the staple products brought in any mountain town in the trans-Missouri country previous to the advent of the railways reflects the ineffectiveness of the transportation. Calico, which was worth 10 cents per yd. in New York, might be worth $3 per yd. in Santa Fé, and a beaver skin was worth 50 cents on the upper Missouri and $3 in New York. As late as 1865 flour sold for a period of several weeks at prices ranging from $75 to $100 per 100-lb. sack in Virginia City, Mont. At the same place, in December, 1864, coal oil was quoted at from $9 to $10 per gal. As late as 1875, potatoes sold for 25 cents per lb. in the San Juan mining camps, and sugar

brought from 35 to 50 cents per lb.* The development of the great trans-Missouri country could come only through the railway.

A writer in *The Emigrant,* a newspaper published for only a few years at Ann Arbor, Mich., in 1832, just three years after the opening of the Liverpool and Manchester road, proposed the construction of a route by rail from New York *via* the south shore of Lake Erie, and the Platte, Snake, and Columbia River Valleys, to the Pacific Coast. The writer of that article made profuse apologies for this proposition, and seemed to fear that his readers would consider him visionary or entirely out of his mind; but through trains on the New York Central, Lake Shore, Chicago and Northwestern, Union Pacific, and the Oregon Short Line traveled the route he then proposed during the lifetime of his contemporaries.

The construction of a Pacific road was frequently mentioned after this, and John P. Davis, in his "History of the Union Pacific Railway," says:

"In 1838, the idea of a Pacific railway had ceased to be novel, and the execution of the project was freely suggested, predicted, and urged in newspapers and magazines."

Bancroft says† that Thomas H. Benton, in a public address in 1844, predicted that men, full grown at that time, would yet see Asiatic commerce crossing the Rocky Mountains by rail.

Though the necessity of a Pacific railway was keenly felt, it is remarkable how many erroneous ideas prevailed in the popular mind in regard to it and the country it was to traverse. Emerson Hough‡ says:

"As late as 1855 it was announced as a settled thing that the continental route could not lie across the Northern Rockies, because in that region the heavy snowfall would block all railway travel."

So general was this opinion that the *Railroad Gazette,* in April, 1870, just after the first winter season in which the Central and Union Pacific had been in operation, announced as something remarkable that the operation of trains on these roads had not been at all impeded by snow during the season. In 1855 Henry Howe (in *The Great West*), expressed the opinion that the route up the valley of the Platte and over the South Pass was wholly impracticable.

* Emerson Hough, in *The Century Magazine*, December, 1901.
† "History of California," p. 500.
‡ In *The Century Magazine*, January, 1902.

"The absence of timber on most of this route would prove an insuperable objection to its selection, even were it not ineligible from other considerations."

That the West itself was to become a great populous country, originating enormous traffic, was apparently not dreamed of at that time. Howe himself says that the country between the "most western limits of civilization and the recently settled territories of the Pacific is confessedly little known."

The Pacific road was to be built, but it was to thrive on oriental trade. The traffic between Europe and China, Japan, Java, and Ceylon, instead of making the weary journey around the southern capes of Africa or South America, was to hurry by rail across America to the cities of the Atlantic seaboard. From these cities, as the world's depots, the products of the Orient were to be distributed to European peoples. From 1840 to 1850 Asa Whitney, a well-to-do New York merchant, seems to have devoted all his time to the agitation of the question of building a Pacific railway, and to him, more than to any one else, is due the creation of the public sentiment which finally brought about the construction of the first Pacific roads. Whitney spent some years in China, and returned to America filled with the idea of constructing a Pacific road to bring the products of the Orient to the eastern coast of the United States. He made examinations and explorations at his own expense, and sent memorial after memorial to Congress. He visited almost every important city and every State in the Union, and went wherever he could get an intelligent body of citizens to listen to his dream, seeking to enlist them in his scheme of getting Congress to make the grants and pass the legislation which would bring about the construction of the Pacific road. His scheme for financing and operating the road was visionary, but, nevertheless, it dominated his whole life in an unselfish way, and brought forth fruit in due time, though never of any benefit to him. It is said that Whitney's entire fortune was spent in this undertaking, and that he was forced to peddle milk in Washington City for a living in the last years of his life.*

During the ten years immediately preceding the Civil War the construction of the Pacific railway was discussed by succeeding Congresses with more earnestness and attention than any other subject

* J. P. Davis, "The Union Pacific Railway," Chicago, 1894, p. 33.

except the slavery question.* All parties were now agreed that a road should be built, but the selection of a route was a question upon which the sections were divided. *De Bow's Review* for May, 1854, said:

"Exploration has convinced every one that there are several ways of connecting * * * the Atlantic and Pacific by ordinary railways. The obstacle to be surmounted is, not the location of a route, but what route to choose of the number already located."

Divided on the issues of slavery, the North wanted a northern route and the South a southern route to the Pacific Coast. When the adherents of the southern route were withdrawn from Congress by the Civil War, federal legislation in the interest of a Pacific railway was at last possible. A Pacific Railroad bill passed Congress in 1862, and the final bill in 1864. The effect of this legislation was to place the eastern terminus of the road at Council Bluffs or Omaha.

In the selection of the eastern terminus of the road, there is evidence of the development of the country while the Pacific railway scheme was taking form. When Whitney first began his agitation, he contemplated constructing the road from Lake Michigan to the Pacific. Later, after Wisconsin had become a State, he advocated beginning the line on the Mississippi at Prairie du Chien.

It is no part of this paper to trace the financial history of this the first transcontinental railway, but it should be pointed out that the form of Government aid extended was an extraordinary piece of national legislation: one which could hardly have prevailed except in such excited times as those incident to the Civil War.

The first Pacific road was built as many a road in the United States has since been built. Long years were spent in deciding whether or not to build, and then construction was pushed with such frantic zeal that much inferior work was necessarily done.† In this railway construction 20 000 men worked as fiercely as though they had been battling for their lives. The results accomplished are perhaps best told by an eye-witness:‡

"There will speedily be other railroads across our Continent. The rivalries of sections, the temptations of commerce, the necessities of our political system, will add at least two more through lines within a generation's time. But this, the first, will forever remain the one

* F. H. Spearman, "The Strategy of Great Railroads." Scribner, 1904, p. 240.
† F. H. Spearman, "The First Trans-Continental Railroad."
‡ Bowles' "Our New West," Chicago, 1870, p. 68.

of history; the one of romance. Its construction in so short a time was the greatest triumph of modern civilization, indeed. The work was seriously begun on the California side in 1864; on the eastern end in 1866; and the early part of the year 1869 witnessed its entire completion. Considering that its length is over eighteen hundred miles; that much of it lay through an almost unknown and unexplored country; that it crosses two great continental ranges of mountains. and almost innumerable sub-ranges; that even the feeble centers of population along the line were five hundred miles apart; that for distances of from two hundred to five hundred miles there was no timber for ties; and for distances of fifty to two hundred miles there was no water for even drinking uses, and no grass to feed animals; that all the iron for one-half of the road had to be shipped around Cape Horn or across the Isthmus of Panama, and for the other to be freighted across the country from Pennsylvania—thinking of these elements of delay, difficulty and cost, surely neither argument nor rhetoric is necessary to place this work in its true proportions and its true relations before the world and in the pages of history."

Under the plan of construction adopted, the Central Pacific, a California corporation, built from San Francisco eastward and the Union Pacific from Omaha westward. The Government bounty money averaged $30 000 per mile, and the companies were enabled to issue and sell an equal amount of first-mortgage bonds of their own. Thus the cash funds available for the construction was nearly $60 000 per mile. Besides this, there remained the capital stock of the two companies and the land grant of half the land for 20 miles on each side of the road, amounting to about 32 000 000 acres. This reward, after the law of 1866 was passed, was open to the rival companies in proportion to the number of miles constructed. The Chicago and Northwestern pushed its lines through to Council Bluffs in the fall of 1867, and thenceforth the Union Pacific had a railroad as well as a river for its base of supplies. Ordinarily, the laying of a mile of track per day on a new line is considered good work, but in the race which ensued nearly 8 miles of track were laid in a single day. In their efforts to push the road, the Central Pacific hauled engines over the Sierras, but their difficulties were greater, as their country on the whole was rougher. They built 850 miles of the road, while the Union Pacific built 1 039 miles.

The two track-laying gangs met at Promontory, Utah, on the north shore of the Great Salt Lake, on May 10th, 1869. The last rail uniting

the Atlantic and the Pacific was laid with great ceremony and nailed down to a laurel tie with spikes of silver and of gold, while the news that the East and the West were at last united by a road for the iron horse was telegraphed far and wide.*

As we have seen, the construction of other transcontinental lines had been contemplated. The United States Government had engineer parties in the field from 1852 to 1854. The famous document, the result of surveys made by these engineers, entitled "Reports of Explorations and Surveys to Ascertain the Most Practicable and Economical Route for a Railroad from the Mississippi River to the Pacific Ocean," included reports on five possible routes, the most northern one of which lay north of the 45th parallel, and the most southern one lay south of the 35th parallel, extending as far south at certain points as the 30th parallel. Since then each of these routes has been very closely followed by a so-called transcontinental road. The most northern route is occupied by the Northern Pacific, the next route south by the Union and Central Pacific, the next by the Denver and Rio Grande, the next by the Atchison, Topeka and Santa Fé, and the most southern by the Southern Pacific.

The Northern Pacific was chartered in 1864, before work was well begun on the Union Pacific. The history of its construction is an account of two decades of strenuous financial struggle.† Construction was begun in 1870, but the road was not completed until 1883. Before this date the Southern Pacific line, which shares with the Santa Fé the right to the term of "transcontinental road," was opened across the continent. The Texas and Pacific was chartered in 1872 to build from Fort Worth, Tex., to San Diego, Cal. While it was delaying construction and making appeals to Congress for aid, the stockholders of the Central Pacific in California, enriched by the liberal aid resulting from the construction of that road, organized in 1874

* "What *is* of consequence is the way in which the clicks [of the telegraph] were received, the blows repeated at San Francisco on the great bell of the City Hall, and cannon booming with the last stroke off Fort Point, and on Capitol Hill in Omaha a hundred guns following the explosion of bombs and the screaming of steam-whistles.

"It was the rejoicing of our crude days. Capitalists, prominent citizens, volunteer firemen, and horseshoers could still walk happily in one tireless procession when the last Pacific Railroad spike was driven Grant took the news in the White House, Chicago turned out a parade four miles long, New York City was saluting the Pacific Coast with salvoes of artillery, Trinity chimes were ringing 'Old Hundred,' and Trinity voices were chanting 'Te Deum' when the earliest transcontinental line was finished; and in Philadelphia the old bells were ringing in Independence Hall." (F. H. Spearman in "The Strategy of Great Railroads," p. 261.)

† E. V. Smalley's "History of the Northern Pacific Railroad." G. P. Putnam's Sons, New York, 1883.

the Southern Pacific Railway Company of California, and consolidated under that name a number of small independent companies in that State. With their own resources they bought and built from San Francisco southeast through Southern California and then east across the Territories of Arizona and New Mexico, along the proposed route of the Texas Pacific, before that company had been able to cross the State of Texas. Through connection was established in 1882 by a junction with the Texas Pacific at a point east of El Paso. By the purchase of the Galveston, Harrisburg, and San Antonio, in 1881, and of the Morgan Railway and Steamship lines, in 1883, the Southern Pacific laid the plans for the consolidation in 1884 of all its allied lines under the name of the Southern Pacific Company of Kentucky.

Contemporaneous with the development of the Southern Pacific was that of the Santa Fé. The Atchison and Topeka was chartered in 1859, and during its earlier years was developed as a Kansas agricultural road. Its lines slowly extended across that State, up the valley of the Arkansas, and finally (in February, 1879) along the Santa Fé trail, over the Raton Spur of the Rocky Mountains. In 1863 its name was changed to the Atchison, Topeka and Santa Fé. It finally joined the Southern Pacific at Deming, N. Mex., thus completing a fourth Pacific route. It was not until after 1888 that the main line of the Santa Fé, crossing north central New Mexico and Arizona, was connected directly with southern California across the scenic Grand Cañon of the Colorado.

The Canadian Pacific was the next transcontinental road constructed. This road shares with the Southern Pacific and the Santa Fé the right to the term, "Transcontinental," as it was constructed from tide-water on the St. Lawrence to tide-water on the Pacific. It lies just north of the United States boundary. This road was a political necessity in Canada, as much as was the Union Pacific in the United States, or more so. Its construction by the Canadian Government was rendered necessary by obligations assumed on the admission of British Columbia to the Dominion. Surveys for the road were commenced in 1871. Construction proceeded slowly until 1880, and then a new company was organized, and the enterprise was pushed to a rapid completion, through to Vancouver, in 1884.

The last so-called transcontinental line to be constructed was the Great Northern, which lies in the territory between the Northern

Pacific and the Canadian Pacific, almost entirely south of the Canadian boundary. Like the Northern Pacific, it now extends from Duluth-Superior, the head of navigation on the Great Lakes, and from St. Paul on the Mississippi, to the Pacific waters at several points in Washington. This was the last of the pioneer roads in the United States. Other roads may afterward be built in wild and inaccessible localities, but, since the construction of the Great Northern, there is no space left where a road can be built for hundreds of miles in practically uninhabited country. The Great Northern was fittingly constructed to close the chapter of pioneer railway building. In many respects its history is unique in the annals of railway construction. It was the first of the transcontinental lines to be constructed throughout by private enterprise, without any form of Government aid, either in land grants or subsidies. Through wise and careful management, it grew by a normal expansion from a local into a transcontinental road. The St. Paul, Minneapolis, and Manitoba was the parent of the Great Northern, and was built to connect the rich valley of the Red River of the North with the Mississippi at St. Paul. The line was extended through North Dakota to the Missouri, building numerous branches north to the Canadian border through the agricultural lands of North Dakota. It was extended west to the Missouri, along that river, and then west and southwest into the rich mining regions of southwestern Montana. The parent company was leased to the Great Northern in 1891.

The decision to extend the road westward to the Pacific Coast was made at about the time when popular interest began to concern itself about the commerce of the Pacific. The extension was begun from Pacific Junction in north central Montana, and the line was constructed rapidly over the Rockies and the Cascade Range and through to Puget Sound. This last of the pioneer lines of the United States was completed to the Pacific Coast in 1893, the year in which the Chicago Columbian Exposition was held in celebration of the four hundredth anniversary of the discovery of America.

The completion of this great work seems to have been signalized only by the kind of celebration which has sooner or later ensued on the completion of each of the pioneer railways, the kind which has meant so much for the progress and development of the United States.

The celebration consisted in an immediate material reduction in transcontinental freight charges.*

The successful construction and management of the Great Northern has been a wholesome refutation of the idea that such enterprises could be carried out in the United States only by using at first a system of over-capitalization which offered false hopes to investors, to be followed by a reorganization which would result in loss to the original stockholders.

In writing of the Great Northern in the *Railway World* of September 10th, 1904, Professor Edward Sherwood Meade says:

"Here is a road which was originally constructed into a desert; whose original territory was almost destitude of population; a road which was forced to create its traffic as it went, and meanwhile to live off its previous accumulations of earnings; a road whose future was clouded in uncertainty. * * * Building up the country as they went they soon made the new mileage self-supporting and able to pay interest and dividends on capital, none of whose proceeds had been diverted from its original purpose of construction and equipment."

The center of population of the United States moved westward by quite regular steps from 1810 to 1890, except for the long stride, from 1850 to 1860, already referred to; but the pioneer work was wellnigh completed in 1890, and from 1890 to 1900 the westward progress of the star on the map was less than half the average distance moved per decade since 1790. From 1900 to 1910 the census reports indicate that it has moved a little farther to the west. The railroads have overtaken the West, and the East and the West are now united. Heretofore the strong young men of the East were wont to go and try their fortunes in the wilds of the West; but now, while Eastern youth still go west for their fortunes, Western youth go east for theirs.

The writer has followed railway development along east and west lines because the making of the country lay in the bringing of the East and the West into one. The traffic of the great trunk lines between Chicago and the Atlantic seaboard, and of the transcontinental lines from the Great Lakes and the Mississippi Valley to the Pacific, is now the pulse which indicates the health of the nation.

The United States, however, owes to its railroads even a greater debt in the development and union of sections separated along lines

* *The Railway Age*, February 24th and June 16th, 1893.

running more nearly east and west. As we have seen, the East and the West were divided by geographical conditions which were overcome by the railroads. The North and the South were divided by political conditions which were overcome by the railroads through the establishment of means of intercommunication which made the South a part of United America.

The development of through railway traffic in the South was very much slower than in the country farther north. This was due partly to the fact that there was no large city on the eastern sea coast south of Baltimore, partly to the fact that the streams in the great interior basin generally flowed south, affording routes for getting produce to the seacoast unhindered by ice throughout the year, and partly to the fact that these streams had acquired such volume by the time they reached the southern part of the United States as to make their bridging very expensive by east-and-west railways.

Even to-day there are only two bridges spanning the Mississippi south of St. Louis, and none south of Memphis. The bridging of the streams in southern Alabama and Mississippi, which was necessary before through lines could be operated between New Orleans and Atlanta, Louisville, Cincinnati, and New York, was a task calling for the best efforts of the engineers of the country, a task which could not be said to have been accomplished satisfactorily before about 1872.

However, local lines were constructed between a number of important points before the Civil War. The Georgia Railroad joined Augusta and Atlanta (171 miles) in 1839. The Central Railroad of that State joined Savannah and Macon (191 miles) in 1840. The Western and Atlantic joined Atlanta and Chattanooga (138 miles) in 1850. The Atlanta and West Point, together with the Western Railway of Alabama, joined Atlanta and Montgomery (175 miles) in 1853. Nashville and Chattanooga (151 miles) were joined by a road of that name in 1854. Memphis and Chattanooga were joined by the Memphis and Charleston (310 miles) in 1857. The Mobile and Ohio was opened from Mobile to Columbus, Ky., on the Mississippi (472 miles) in 1859, and in the same year the New Orleans, Jackson, and Great Northern, together with the Mississippi Central, made up a continuous line of 442 miles from New Orleans to Jackson, Tenn. The Louisville and Nashville was also opened between the cities named (187 miles) in 1859, and between Louisville and Memphis by the addition

of 262 miles more of road in 1861. The Mobile and Montgomery was opened from Montgomery to Tensas (164 miles), within 22 miles of Mobile by water, in 1862.

The first connection of continuous railway line penetrating the interior of the South and putting it in direct and quick communication with the country's Capital and the cities of the Atlantic seaboard may be said to have started at Alexandria, just across the Potomac from Washington; it crossed the State of Virginia diagonally to the southwest, and followed the general direction of the Blue Ridge Mountains into Alabama, Georgia, and eastern Tennessee. Though there were few through lines operated, rail connection was quite well established through the territory mentioned above by 1859.

During the Civil War, and for many a sad, long year afterward, railway development was very nearly at a standstill in the South. In 1860 30% of the railway mileage of the country was in the Southern States. In 1870 only 23% of railway mileage was in the South. From 1860 to 1870 railway mileage increased 91% in other parts of the country and only 35% in the Southern States. By the time railway expansion began again, the economies to be gained by consolidating separate lines into continuous systems had been pretty well demonstrated by the lines farther north.

The Illinois Central was the first exterior road to push its lines into the South. This road, extending from Chicago and the Upper Mississippi to the mouth of the Ohio at Cairo, had been of great service to the Northern armies in the transportation of troops. Paralleling the Mississippi, it had found that it could compete successfully with the river, even for heavy traffic. In 1872 steps were taken to interchange traffic across the Ohio at Cairo with the two roads already named extending from New Orleans to Jackson, Tenn., and on December 24th, 1873, through traffic was opened all the way from Chicago, the metropolis of the West, to New Orleans, the metropolis of the South.

From the date of this peaceful invasion of Southern territory by a Northern road, two great changes were to be made in conditions which had hitherto materially affected the relations of the South with the remainder of the country. One of these changes was to be regretted, perhaps, while the other was most propitious. The immense steamboat traffic on the Ohio and the Mississippi, of which the

South was justly proud, was to be curtailed in the same manner as had been the traffic on the Erie Canal; and there was to come a happy end to the narrow local prejudices which had so long surrounded so many Southern communities.

At about the same time the Louisville and Nashville—a rival system—began to extend its lines from the Ohio River through the South. It reached down to the Tennessee River in Alabama by acquiring the Nashville and Decatur in 1872, and in the same year to the Alabama River at Montgomery by completing and controlling the South and North Alabama, which line was later to develop the wonderful coal and iron fields in the Birmingham district. It reached tide-water at Mobile in 1880 by acquiring the Mobile and Montgomery, and then in the same year joined the Illinois Central at New Orleans by absorbing the New Orleans and Mobile. By 1880 through lines of railway traffic and travel had been well established from New Orleans to Chicago *via* Memphis and Cairo; from New Orleans to the East, *via* Birmingham, Cincinnati, and Pittsburg, by both the Louisville and Nashville and the Queen and Crescent route over the famous Cincinnati Southern; by the Shenandoah Valley route *via* Montgomery, Atlanta, Chattanooga, and the Blue Ridge Mountains; and *via* the Piedmont Air Line through Montgomery, Atlanta, Charlotte, Greensboro, Danville, and Washington.

Instrumentalities by Which the Development of Railways Has Been Fostered.—Even this brief survey of pioneer railway development would be incomplete without some mention of at least two of the means of development without which railway construction must have waited many years: One of these, the land grant, directly affected railway construction; and the other, the steel rail, directly affected railway operation. Without either one of these improvements, railway development would probably have been retarded a decade or more behind its present-day status.

The first land grant was made in September, 1850, with a view of establishing a trunk line from the Michigan to the Gulf of Mexico; it was received by the Illinois Central. On the whole, 155 000 000 acres have been donated to railways of encouraging railroad construction land grants, for the purpose and Territories west of the Mississi The railways of all States means. The railways of Alabama a were liberally aided by this all the States on the eastern

shore of the Mississippi, except Tennessee and Kentucky, were more or less indebted to land grants for their development.

Some account of the troubles experienced with the early forms of iron rails has already been given. Space is lacking to enter into a history of the development of the form and manufacture of the steel rail, but it may be mentioned that it is due to the homogeneity of steel that the rails of the present day finally fail largely by normal wear, or the abrasion of the rail head, rather than by splitting and lamination, as was the case with iron rails.

Steel rails were rolled in England in 1855, but after the process of manufacture had been introduced and improved in the United States, the first orders for steel rail were placed in 1867. The following record, compiled from Poor's "Manual," shows the rate at which steel rails supplanted those of iron:

Year.	Miles of track with steel rails.	Miles of track with iron rails.	Percentage of steel rails.
1867	.....	.....	00
1880	33 680	81 967	29.1
1885	98 102	62 495	61.0
1890	167 606	40 697	80.4
1895	206 546	28 652	87.8
1900	238 464	19 389	92.5
1903	271 013	15 249	94.7

There is no doubt that, since about 1898, the use of iron rails has been practically discontinued, except on the tracks of sidings and spurs infrequently used.

The greatest service the steel rail has rendered has been in permitting the use of heavier rolling stock and in adding to the safety of train operation, so that traffic could be moved at greater speed and with fewer interruptions; but it has also introduced great economies in the railway maintenance account. Comprehensive and reliable statistics of railway operating expenditures in the days of iron rails are not available, but, from the best records that could be collected by the late A. M. Wellington, M. Am. Soc. C. E., writing in 1887,* in the ten years from 1865 to 1875, the renewal of rail made up 28.9% and the renewal of ties 11.7% of the total expenditures for maintenance of roadway and structures. Statistics of railways show that, for the average of the three years, 1907-09, rail renewals made up 5.1% of this account and tie renewals 14.7 per cent. When iron rails were used they cost 2.5 times as much as ties. To-day, with

* Wellington's "Railway Location." New York, 1889, p. 120.

steel rails, the ties cost 2.9 times as much as the rails. However, these ratios between rail and tie accounts do not measure accurately the economy in maintenance produced by the steel rail, because the actual and relative cost of ties has greatly appreciated on account of the diminishing timber supply.

DISCUSSION

Mr. Pugh.

MARSHALL R. PUGH, M. AM. SOC. C. E. (by letter).—In his account of the early development of railroads in the United States, the author refers to the tramway at Quincy, Mass., constructed in 1826, as being the first. This tramway, however, was antedated by one in Delaware County, Pa., which was built by a Scotch millwright named Somerville, under the direction of John Thomson, a civil engineer, in September, 1809. This was an experimental piece of track, 60 yd. long, executed for Thomas Leiper, of Philadelphia. It had a gauge of 4 ft., a grade of 4 per 100, and the rails, of which the writer has no details, were carried on sleepers spaced 8 ft. apart. A loaded car was run on this track so successfully that Mr. Leiper built a road about 1 mile long, running from his quarry on Crum Creek to his boat landing on Ridley Creek, in Delaware County, and this road continued in successful operation for nineteen years.*

In 1815 a law was passed by the New Jersey Legislature authorizing Colonel John Stevens to construct a railroad from Trenton to New Brunswick, but nothing was done. On March 31st, 1823, Colonel Stevens secured a charter from the Pennsylvania Legislature to construct a road from Philadelphia to Columbia, on the Susquehanna, Stephen Girard being one of the corporators. This anticipated the present charter of the Pennsylvania Railroad, but it was repealed in 1826, before anything had been accomplished.

During 1826 and 1827 the Delaware and Hudson Canal Company started to build lines to bring anthracite coal from the Susquehanna Valley across to tide-water, employing a young civil engineer, Horatio Allen.† Mr. Allen writes:‡

"Early in the year 1827, I had given all the attention that it was in my power to give, and having come to conclusions as to the locomotive, that all subsequent experience has confirmed, and believing that the future of the civil engineer lay in a great and most attractive degree in the direction of the coming railroad era, I decided to go to the only place where a locomotive was in daily operation, and could be studied in all its practical details."

Mr. Allen left the Delaware and Hudson Canal Company preparatory to going to England, and the Company decided to intrust him with the commission of securing for them from England their railroad iron and three locomotives, on plans to be approved by him. Thus the first order for a locomotive, after the success of the Stockton and Darlington engine, came from an American company on the advice of an American civil engineer.

* Sipes', "The Pennsylvania Railroad," Philadelphia, 1875.

† Mr. Allen was subsequently the Fifth President of the American Society of Civil Engineers.

‡ "The Railroad Era," p. 13, New York, 1884.

Mr. Pugh. Mr. Taylor has referred to the failure of the Delaware and Hudson locomotives as being due in part to their not having Stephenson's last improvement, the multitubular boiler; but Mr. Allen* writes quite at length:

"As to the boiler, the result on my mind was a decided confidence in the multitubular boiler proposed by Mr. Booth, of the Liverpool and Manchester road, but I found in many a distrust of that plan of boiler as being an *untried* boiler. John U. Rastrick, of Stourbridge, * * * recommended a boiler of small riveted flues of as small a diameter as could be riveted, and in number as many as the end of the fire-box would allow."

Mr. Allen ordered two locomotives from Stephenson and one from Foster, Rastrick and Company, of Stourbridge.*

"The only points decided by me were that the boilers of the locomotives built by Stephenson & Co. were to be multitubular boilers; and that the boiler of the locomotive built by Rastrick & Co. (the 'Stourbridge Lion') was to be a flue-boiler."

These engines reached New York in the winter of 1828-29, and in the spring, the *Stourbridge Lion,* which owed its name to a fierce and resplendently colored lion painted on the end of the boiler, was brought by river and canal to Honesdale, Pa. It is not known how this engine came to be sent, but it is unfortunate that one of Stephenson's was not chosen, as they were the prototypes of the *Rocket* whose epoch-making trip took place in October of the same year.

That first trip of a steam locomotive on the American Continent is best told in the words of the man who made it, Mr. Allen himself driving the engine.†

"The line of road was straight for about 600 feet, being parallel with the canal, then crossing the Lackawaxen Creek, by a curve nearly a quarter of a circle long, of radius 750 feet, on trestle-work about thirty feet above the creek, and from the curve extending in a line nearly straight into the woods of Pennsylvania.

"The road was formed of rails of hemlock timber in section six by twelve inches, supported by caps of timber ten feet from centre to centre. On the surface of the rail of wood was spiked the railroad iron—a bar of rolled iron two and a quarter inches wide and half an inch thick.

"As the locomotive was seen in mid-air, in its transference from the canal to the railroad, the opportunity was had to see that the axles had an unyielding *parallel* position, there being no king-bolt that provided facility for passing round the curve, and that therefore, the four wheels holding their rigid position were to be forced round the curve by the power of the steam-engine. The locomotive

* "The Railroad Era," p. 17.

† *Ibid*, p. 20 *et seq.*

FIG. 1.—"OLD IRONSIDES."

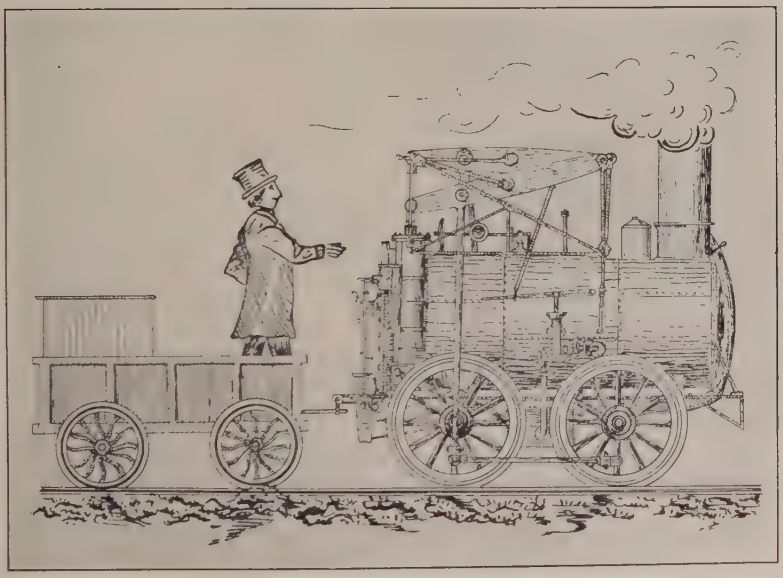

FIG. 2.—THE "STOURBRIDGE LION."

thus seen altogether impressed the lookers-on as being of great weight. The road having been built of timber in long lengths, and not well-seasoned, some of the rails were not exactly in their true position when the time came that they were to carry the locomotive in its onward movement. Mr. Pugh.

"Under these circumstances the feeling of the lookers-on became general, that either the road would break down under the weight of the locomotive, or, if the curve was reached, that the locomotive would not keep the track, and in its onward motion without support it would dash into the creek with a fall of some thirty feet.

"On my part, I knew that the road would carry the locomotive safely, and that the curve would be passed without any difficulty.

"But when the time came, and the steam was of the right pressure, and all was ready, I took my position on the platform of the locomotive alone, and with my hand on the throttle-valve handle said: 'If there is any danger in this ride it is not necessary that the life and limbs of more than one should be subjected to that danger; that, having no doubt whatever, I was about to take the ride entirely alone, and that the time would come when I should look back with great interest to the ride that was now before me.'

"The locomotive, having no train behind it, answered at once to the movement of the hand, and there being no doubt as to the result, a motion was had at once in which there was not any evidence of distrust; soon the straight line was run over, the curve was reached and passed before there was time to think as to its not being passed safely, and soon I was out of sight in the three miles' ride alone into the woods of Pennsylvania.

"I had never run a locomotive nor any other engine before, I have never run one since; but on that 9th of August,* 1829, I ran that locomotive three miles and back to the place of starting, and being without experience or a brakeman, I stopped the locomotive on its return at the place of starting. After losing the cheers of the lookers-on, the only sound, in addition to that of the exhaust steam, was that of a timber structure when the parts are brought into the bearing state."

At this time Mr. Allen had just received his appointment as Chief Engineer of the South Carolina Railroad, and though the Baltimore and Ohio was now under construction, the South Carolina line, completed in 1835, from Charleston, S. C., to Hamburg, opposite Augusta, Ga., a distance of 135 miles, was the longest railroad in the world. With this length of line, night operation of trains became a necessity. The expedient adopted by him for the first locomotive headlight is rather amusing.†

"That the locomotive was to be used in the night, and during the whole night, was plainly to be anticipated. It was thought well to make trial of such running by night, that it might be known what it was necessary to provide. For such trial two platform cars were placed in front of the locomotive. On the forward platform was placed an

* Contemporary records seem to show that it was the 8th of August.

† "The Railroad Era," p. 28.

Mr. Pugh. inclosure of sand, and on the sand a structure of iron rods somewhat of urn shape. In this structure was to be kept up a fire of pine-wood knots. Suitable signals as to the rate of speed, etc., were provided. The day preceding the evening of the trial closed in with as heavy a fog as I have ever seen, and I have seen a first-class London fog. But the fog did not prevent the trial when the appointed time came.

"The country to be run through was a dead level, and on the surface rested this heavy fog; but just before we were ready to start, the fog began to lift and continued to rise slowly and as uniformly as ever curtain left surface of stage, until about eighteen feet high; there it remained stationary, with an under surface as uniform as the surface it had risen from. This under surface was lit up with radiating lines in all directions with prismatic colors, presenting a scene of remarkable brilliancy and beauty.

"Under this canopy, lit on its under surface, the locomotive moved onward with a clearly illuminated road before it; the run was continued for some five miles, with no untoward occurrence, and I had reason to exclaim, 'The very atmosphere of Carolina says, "Welcome the locomotive."'"

At about the same time, 1834, the State of Pennsylvania had a line extending from Philadelphia to Pittsburg, consisting in part of railroad and in part of canal. The first 82 miles from Philadelphia to Columbia was rail; then came 172 miles of canal to Hollidaysburg. The Portage Railroad, 36 miles in length, extended to Johnstown, and from there there were 104 miles of canal to Pittsburg, a total distance of 394 miles. It was operated by horses until 1836, when locomotives were put on. Thereafter the State furnished the motive power, while the cars were the property of individuals, who paid regular rates of toll for being drawn by the engines, and for the use of the tracks.

The Germantown and Norristown Railroad, from Philadelphia to Germantown, is interesting as having had the first successful American locomotive, *Old Ironsides*. The Delaware and Hudson engines were English, and moreover failed to satisfy American conditions. Peter Cooper's locomotive, the *Tom Thumb*, was an experiment only; the *Best Friend*, and the *West Point*, built at Cold Spring, N. Y., for the South Carolina Railroad, and the *DeWitt Clinton*, built at the same place for the Mohawk and Hudson Railroad, were like the English engines, with rigid axles, and did not meet American needs. The Germantown and Norristown road was laid out under the direction of its Chief Engineer, J. Edgar Thomson, subsequently Chief Engineer, and then President, of the Pennsylvania Railroad. It was opened for use with much ceremony on June 6th, 1832, the event being one of such importance that among the guests were the Councils of New York City. The cars, or carriages, made in Baltimore, were drawn by horses until November 22d, when *Old Ironsides*, Baldwin's first locomotive, made her trial trip. This engine, the first

product of the Baldwin works,* weighed more than 5 tons. The spokes of the wheels were of wood, as were the rims, the tires being of wrought iron and the hubs of cast iron. Such a marvel was this machine that the President of the Railroad Company, fearing that some fatal accident might occur, usually rode on the train on its trips. Notice was given that when it rained horses would draw the cars, as the engine was not taken out in the wet. Mr. Pugh.

Schedules were not strictly adhered to. An old gentleman has stated that he went to the station intending to ride out on the train, and, though he was early, he was told that it had started quite a little while before. The ticket agent said that the Directors were to feast in Germantown, and, to avoid the dire disaster of a cold dinner, had started the train half an hour ahead of the proper time.

The earliest engines had no whistles, signals being given by pushing up the safety valve and allowing the steam to escape with a noisy hiss. Loose chain couplings and the entire absence of brakes gave rise to a most exhilarating series of jolts and jerks in starting and stopping. On the New Castle and Frenchtown Railroad, when the signal of an approaching train was heard, the slaves around the station would rush up to it and seizing hold of the cars, drag back on them with might and main, while the ticket agent stuck a stout stick through a wheel, and the whole train was thus gracefully brought to a standstill.

Some roads in the Cotton Belt had a flat car, piled with bales of cotton, placed between engine and train—a so-called barrier car. In event of a boiler explosion, it was thought that the passengers would be protected, and in case of collision they would simply be pitched head first into a sort of mattress or soft cushion. The writer has no reports on its operation under "service conditions."

R. L. Morrison, Jun. Am. Soc. C. E. (by letter).—The first railway mentioned in the paper is the line from Wandsworth to Croydon, which was chartered in 1801. This was the first chartered railway, but it may be of interest to note that the first English railway of which there is any record seems to have been located at Newcastle-on-Tyne, at the coal mines, and is described by Roger North, who saw it in operation in 1676. He says:† Mr. Morrison.

"Another thing that is remarkable, is their way-leaves. * * * The manner of the carriage [of coal] is by laying rails of timber, from the colliery, down to the river, exactly straight and parallel; and bulky carts are made with four rowlets fitting these rails; whereby the carriage is so easy that one horse will draw down four or five chaldrons of coals, and is an immense benefit to the coal merchant."

* Mr. Baldwin had so much trouble in constructing this engine that he vowed he would never make another. How well he kept his vow is shown by the fact that in the early part of 1907, the Baldwin works had turned out its 30 000th locomotive.

† "Lives of the Norths," Pt. I, pp. 281–282.

Mr. Morrison.

A Newcastle "chaldron" is somewhat more than 72 bushels, heaped up, or 5 300 lb. Robert Stephenson says* these "way-leaves," or railways, were in use before 1671.

For many years these railways were used only at mines, and had wooden rails. In 1767 the scarcity of timber in England led to the introduction of solid rails of cast iron, which were supported at the joints by blocks, first of timber and afterward of stone. The first rails were flat, but in 1789 "edge-rails" were introduced.†

Although Mr. Taylor does not state that the locomotive built by Stephenson in 1814 was the first ever constructed, the reader might easily get that impression. In 1801 Oliver Evans, of Philadelphia, constructed a dredging machine which was mounted on wheels and conducted under its own steam through the streets of the city to the Delaware, a mile distant.‡ Evans' one contribution to science was the non-condensing, high-pressure, steam engine which impelled this strange device.§

The first English locomotive was designed by Richard Trevithick, a Cornish mine engineer, in 1804, and was successfully operated on common roads.‖ Trevithick built an improved locomotive in 1808.

In 1813, William Hedley and Timothy Hackworth succeeded in constructing the first locomotive which was run on rails. This was called the *Puffing Billy,* and was an engine with two vertical cylinders. George Stephenson, who was employed at a neighboring colliery, built a locomotive closely following that of Hedley.**

The tramway built at Quincy, Mass., in 1826, is mentioned by Mr. Taylor, as being the first tramway in America. Cleveland and Powell, in discussing tramways, state:††

"The earliest work of the kind in this country appears to have been constructed by Silas Whitney, of Boston, on Beacon Hill, in 1807. Two years later, Thomas Leiper built a tramway to connect his quarries on Crum creek, Delaware county, Pennsylvania, with tide-water on Ridley creek, a distance of about three-quarters of a mile. This was in constant use until 1828. Other similar structures were laid down at Falling's creek, Chesterfield county, Virginia, soon after 1810; at Bear Creek Furnace, Armstrong county, Pennsylvania, in 1818; and at Nashua, New Hampshire, in 1825."

Mr. Boucher.

William J. Boucher, Assoc. M. Am. Soc. C. E. (by letter).—Mr. Taylor has prepared a very interesting paper on early railways, and deserves credit for his painstaking account; the thanks of the Engineering Profession are due him for thus recording these important historical facts in such a readable manner.

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* "Life of Robert Stephenson," p. 115.

† Cleveland and Powell, "Railroad Promotion and Capitalization."

‡ *New York Review*, III, pp. 95–96.

§ Thurston, "History of the Steam Engine," p. 153.

‖ Niles, III, Addenda, p. 5.

** Sinclair, "American and Foreign Locomotives," New England Railroad Club, *Proceedings*, January, 1902.

†† "Railroad Promotion and Capitalization," p. 60.

Mr. Boucher.

The writer desires to add to the history, and, at the same time, pay deserved tribute to the Stevens family, of Hoboken, N. J., founders of the Stevens Institute of Technology, a family which has stood at the very forefront of engineering achievement since the beginning of the Nineteenth Century. John Stevens and his sons, Robert L. and Edwin A., possessed to a marked degree those characteristics which distinguish the modern engineer; their ability to solve problems and forecast the future was indeed ahead of their day.

Long before any locomotive was constructed in Europe, John Stevens had clearly worked out, in his own mind, the theory of such an application of steam, and the actual form in which it could be advantageously made, as well as the cost of constructing and working a railway for the use of locomotives. Long before there was any experience to justify his anticipation, he said and published, "that there was no limit to the speed of a locomotive on a rail, but the strength of the materials, indeed it might easily be made to run as fast as a pigeon could fly;" and it is one of the striking incidents connected with the early days of the Camden and Amboy Railroad (now part of the Pennsylvania system in New Jersey) that a flock of pigeons, which had settled on the track, being disturbed by the rapid approach of the engine, took wing in the direction the train was moving, and one of them, attempting to cross in front of it, was struck down, thus most literally verifying the prediction made.

In 1810, while discussion was going on relative to the project of connecting Lake Erie with the Hudson River, Col. John Stevens, after seeing the report, which contemplated a canal with locks, earnestly pressed upon the Commissioners the consideration of a system of railways adapted to the use of carriages propelled by steam locomotives, as being preferable in economy, speed, and rapidity of construction. In 1812, he published his views in a pamphlet entitled "Documents Tending to Prove the Superior Advantages of Rail-ways and Steam Carriages Over Canal Navigation," which is practically a copy of his memorial to the Commissioners, with their objections and his rejoinders. In it he writes of the advantages to the public and to the Federal Government in the collection of tolls and in the protection of distant points from invasion by an enemy, and again refers to the possible speed, which might equal 100 miles per hour, but considers it probable that it may not in practice be convenient to exceed 20 or 30 miles per hour.

In an address delivered by the late J. Elfreth Watkins, Assoc. Am. Soc. C. E., before the Philosophical Society of Washington, D. C., on May 7th, 1892, on "John Stevens and His Sons, Early American Engineers," he said:

"The South Carolina Railroad (commenced in 1829) which, when completed in 1832, was the longest railway in the world, was constructed upon his plans, as laid down 20 years before."

Mr. Boucher.

In the early part of the last century, the principal route of traffic between New York and Philadelphia was by water from New York to New Brunswick, then overland to Trenton, then down the Delaware River to Philadelphia. John Stevens was associated with the firm which controlled this route, and he, seeing the advantages which would be possessed by a rail line over the wagon roads, in 1817 obtained a charter from the State of New Jersey, "to build a Railroad from the river Delaware, near Trenton, to the river Raritan, near New Brunswick." This is undoubtedly the earliest railroad charter granted in America, but no tangible result followed, because the scheme was regarded as wild and visionary. Col. Stevens regretted that his financial condition was not such as to warrant him in building the road at his own expense. His interest in the subject did not flag on account of this failure, for, in 1823, through his exertions, acts were passed by the Legislature of Pennsylvania for the incorporation of a company to construct a railway from Harrisburg to Pittsburg, and another company to construct a railway from Philadelphia to Columbia, in Lancaster County, among the incorporators being John Stevens, Stephen Girard, and Horace Binney.

In 1826, Col. Stevens, then 76 years of age, constructed at his own expense a locomotive with a multitubular boiler, which he operated for several years on a circular track on his estate at Hoboken.

"This was the first locomotive in America driven by steam upon a track, of which there is any reliable record."*

A model of this locomotive, together with the original multitubular boiler which formed a part of it, is preserved in the National Museum at Washington.

On April 12th, 1830, was held the first meeting of stockholders, at which the organization of the Camden and Amboy Railroad was effected. The following persons were chosen the sole officers and directors of the company: Robert L. Stevens, of Hoboken, President; Edwin A. Stevens, of Hoboken, Treasurer; Jeremiah H. Sloan, of Camden, Secretary. Robert L. Stevens was also appointed Chief Engineer. The brothers, Robert L. and Edwin A. (with their father, John Stevens), became the active managers of the road. The former took charge of laying the track and procuring the equipment, while the latter looked after the political, financial, and practical management of affairs. Considerable judgment had to be used in both these branches. No laws had been established in regard to the steps to be taken in the purchase of right of way, and many of the New Jersey legislators were very chary when they were asked to grant franchises and rights. In the matter of construction and equipment, these early engineers had little or no experience to guide them, and they had to resort to experiment and trial.

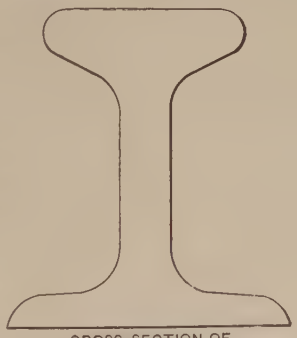
* J. E. Watkins, Address at Washington, D. C., May 7th, 1892.

Mr. Boucher.

The road, the first link of the present Pennsylvania system, was completed and formally opened for traffic, from Bordentown to Hightstown, N. J., on October 9th, 1831. In December of the same year it was opened to Amboy, but locomotives were not used until August, 1833, when a sufficient number had been built.

Robert L. Stevens advocated the use of an all-iron rail, in preference to the wooden rail plated with strap-iron then in use on some of the American railways. At a meeting of the board of directors of the Camden and Amboy Railroad, a special resolution was passed, authorizing him to obtain the rails he advocated. At that time, there was no mill in America capable of rolling such rails as he had in mind, therefore, soon after this, he sailed for England. On the voyage, he passed hours in whittling thin wood into shapes of imaginary cross-sections, until he finally decided which one was best suited to the needs of the new work. He was familiar with the Birkenshaw rail, with which the best English railroads were then being laid, but he observed that as it required an expensive chair to hold it in place, it was not adapted to conditions in America, where metal workers were scarce and iron was high in price.

He devised the base to the T-rail, thus dispensing with the necessity for the chair. He also designed the hook-headed spike (which is substantially the railroad spike of to-day), and the "iron tongue," which has developed into the fish-plate or angle-bar, and the rivets (which have been replaced by bolts and nuts) to complete the joint. Thus, with his all-iron rails, spiked to wooden ties, he first produced the type of track which is in general use throughout America at the present time.



CROSS-SECTION OF ROBERT L. STEVENS' FIRST T-RAIL.

FIG. 2.

Much difficulty was experienced in obtaining a mill to roll the newly-designed rail, for not only were no rolls in existence and the cost of their making was large, but there was also much doubt as to the practicability of rolling such a shape. An iron-works in Dowlais, Wales, however, was prevailed upon to attempt the work, Mr. Stevens being required personally to guarantee payment for damages to the rolling machinery in case of breakdown. On coming from the rolls, the rails were crooked and twisted, but, by persistence, he showed the mill men how to straighten them while they cooled. The first rails weighed 36 lb. per yd., but, very soon, the weight was changed to about 40 lb. per yd., and they were rolled in 16-ft. lengths, the latter rails being $3\frac{1}{2}$ in. high, $2\frac{1}{8}$ in. wide at the head, and $3\frac{1}{2}$ in. wide at the base. With the exception of slight

Mr. Boucher. alterations in the proportions, incident to the increased weight, the rail, as designed by Mr. Stevens and first used on the Camden and Amboy track, is the rail of to-day. The price of these rails, in England, was £8 per ton, and the import duty was $1.85 per ton.

On the occasion of the opening of the Carnegie Laboratory of Engineering at Stevens Institute, on February 6th, 1902, a piece of one of these original rails was presented to Mr. Andrew Carnegie, with appropriate ceremonies, and accompanied by the documentary proofs of genuineness.*

Mr. Hammond. GEORGE T. HAMMOND, M. AM. SOC. C. E. (by letter).—The student of engineering history, who seeks, in the publications of this Society, for data concerning interesting structures which, at an early day in our art, have been of importance in railway development in the United States, will find one item which will strike him as curious, and will prove somewhat of a mystery, if he attempts to find out much about it.

The writer refers to the picture† entitled "Tunnel under Part of Atlantic Avenue, Brooklyn (from a Crayon Sketch)." Examining this picture, he will see the entrance of a horse-shoe shaped tunnel structure for a two-track railroad, about 21 ft. wide and 17 ft. high, out of which issues a train drawn by a locomotive engine of very early design, and he will observe the legend at the top of the picture "Atlantic Ave. Tunnel, 1845." The text states that this tunnel was built by the Long Island Railroad Company in 1845, and operated under a portion of Atlantic Avenue, but later it was filled up.

As this tunnel was the first subway in New York City, and was in many ways a great undertaking at the time, it is of no little interest. The writer, having often come into contact with this work, and once at least into physical contact with it in the progress of sewer construction, of which he then had charge, had always found in it an interesting mystery. Why it was built, and why abandoned, and who might have been the Chief Engineer under whose supervision it was designed and erected, had always eluded him, until he chanced to see the statement, in Prime's "History of Long Island," published more than fifty years ago, that the Chief Engineer of the Long Island Railroad was James Shipman. Subsequent inquiry, following this clue, developed the interesting fact that James Shipman, or more correctly, James Ingraham Shipman, at one time a member of this Society,* who was Chief Engineer of the Long Island Railroad from about 1837 to 1847, was a man who contributed to railroad development in the United States, and at one time stood near the head of his

* The foregoing is largely taken from the "Morton Memorial Volume," published in 1905 by the Alumni Association of the Stevens Institute of Technology.

† *Transactions*, Am. Soc. C. E., Vol. LXVIII, Plate IV, p. 36.

* In the List of Members for 1871 his name is printed erroneously J. J. Shipman, but in the 1873 List it is given correctly, James I. Shipman.

profession, whose memory has been nearly, if not quite, forgotten. The few facts which the writer has been able to gather concerning him were obtained from his surviving son, Mr. James T. Shipman, of Farmingdale, Long Island, himself a civil engineer, who has for many years followed in the footsteps of his father as a railroad builder. Mr. Hammond.

The Long Island Railroad, incorporated in 1834, was one of the first roads constructed in America, and must be given an important place in the progress of railway development. It was almost our first road, and none of our railroads antedated it more than five or six years. The purpose of this enterprise was to provide a main line between New York and Boston, by railroad to the east end of Long Island, and thence by steamboat to Boston. In the early days of passenger traffic, this was much the quickest way to reach the latter city from New York; later, the time was further reduced by a steamboat connecting with the Providence and Stonington Railroad and the Boston and Providence Railroad.

In 1844 the railroad was completed to Greenport, and during the following year the tunnel under Atlantic Avenue, Brooklyn, was built, greatly improving the grades between Bedford Station and South Ferry. The Long Island Railroad was constructed for what in those days was considered extremely high speed. It is recorded that the trip from South Ferry, Brooklyn, to Greenport was made in $3\frac{3}{4}$ hours, and that it had been accomplished in the earliest days of the road in 2 hours and 35 minutes "with perfect safety," the distance being 95 miles. The railroad was completed to Hicksville, and trains were run thereon from South Ferry, Brooklyn, in 1837. Thence it was extended to Greenport during 1841-1844.

After the completion of the main line of this railroad, Mr. Shipman left its service, but returned to it as Chief Engineer after an interval of about ten years.

On leaving the Long Island Railroad, Mr. Shipman removed to Illinois, where, during the period of 1847 to 1855, he was Chief Engineer of the Sangamon and Morgan Railroad and of the Alton and Sangamon Railroad; also of the Illinois Coal Company's Railroad, etc.

While Chief Engineer of the Alton and Sangamon Railroad (1852) he prepared a manual, published by that railroad company, entitled "Facts and Figures Illustrative of the Alton and Sangamon Railroad; with a Report on the Extension to Bloomington, Pekin, and Peoria." This curious publication, placed in the writer's hands by Mr. Shipman's son, affords such an insight into the early development of the art of railroad building, that a few extracts from it will be given.

It should be noted that the Alton and Sangamon Railroad was destined to become one of the main carriers of traffic in America,

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between Chicago and St. Louis. In clear and cultivated English, which might well be studied as a model in these days, Mr. Shipman tells of the problems solved in railroad building. He recommends that all bridges be built of iron; possibly this is the earliest recommendation for the use of this material in railroad bridges (1852). It would appear so, as he does not refer to a single instance of an iron bridge in use or recommended for this purpose, yet he seems to be quite familiar with the practice of that time, and refers to the great railroad enterprises built and projected as though he was fully informed. The mention of a single iron bridge for railway service would have strengthened his recommendation.

To select extracts from an original document, the whole of which is of interest, is not easy. The extracts which follow are but a small portion of the whole, but are general, and serve to illustrate the art of railway building sixty years ago.

"The Alton and Sangamon Railroad Company * * * chartered in the year 1847, * * * when completed will connect the capital of the State of Illinois with the Mississippi river, at its nearest practicable point, by a line of about seventy miles. The Road is to be an exceedingly important one, both from its local business, and from its connection with other roads. As a *local* Road, it must always constitute the outlet for the Central portions of Illinois, to the Mississippi river. The great fertility of this portion of the State is well known; and though Illinois, as a whole, is very sparsely populated, the portions traversed by the above line will compare favorably with the best settled parts of Indiana and Ohio, and are not excelled, if equalled, by either in extent of natural resources.

* * * * * * *

"In looking at the prospective business of Western Roads, we believe a much more correct estimate can be found of its extent, from a general view of the productiveness and course of trade in that section, than from the most elaborate compilation of statistics drawn from existing data. The amount of production, and extent of movement of property, is now subordinate to the *means* of transportation. Some of the most fertile portions of Illinois are valueless, simply for the want of a suitable outlet to a market. Many sections, that are destined to be the most flourishing, are still covered with forests. In many portions of the State, the surplus of grains raised will not bear the cost of carriage to navigable waters. The rivers of Illinois, the present route of commerce, though invaluable in the absence of Railroads, are for a considerable portion of each year not available for transportation. From the commerce that has already been developed, we can form but a faint idea of what is to come, when the suitable avenues and instruments shall have been provided.

* * * * * * *

"We find our own views of the extent and nature of the business on Western Roads, and the estimates which we have given above, strikingly confirmed in a recent pamphlet, put forth by D. A. Neal, Esq., one of the corporators of the Illinois Central Railroad, upon the prospective business and income from that great work. As Mr. Neal

is admitted to be the very highest authority in reference to all matters touching Railroads, we propose to compare, as far as may be done, his estimates, with those which we have prepared in reference to the Alton and Sangamon Railroad, as far as a parallel between the two holds good. Mr. Hammond.

"The *first* item in Mr. Neal's estimate of the business of the Central Railroad, embraces the articles of wood and coal. As our estimates fall below his, we keep our figures of revenue from these sources, viz.: $22,800.

"2nd. The second item in his estimate is 'the produce of the soil.' Upon this he says:

" 'The second source indicated, from which the Road is to be sustained is the "produce of the soil." The transportation of Indian corn and other grain *will be the important business of the Road.* Their production will in time be limited by the capacity of the Road to carry it off. In an estimate of this sort then, it may be safe to neglect all other kinds of produce, or rather consider them as merged into the one article of maize or Indian corn. We have already restricted ourselves to a population in the district of country lying within fifteen miles of the Railroad, to the density of the whole State, by the census of 1850, or fifteen to the square mile. There will be, as before stated, 20,000 of such miles, and the number will of course be 300,000. A large portion of these will be male adults; but taking the usual calculation of five to a family, we have 60,000 families. Deducting again one-sixth for other employments, we have then 50,000 families presumed to be in the cultivation of the soil. Now it cannot be doubted, I think, that ten families can easily till and take care of 1 000 acres of Indian corn in Illinois, by an interchange of labor. It is as certain that the land will give 50, 60, or 70 bushels to the acre. This gives an average of 100 acres of cultivated ground, and 6,000 bushels of corn to each family. But to be sure not to overtask the powers of the people, or of the soil, we reduce both one-half, 50 acres and 30 bushels per acre, giving to each family, 1,500 bushels. Of this, suppose one-third to be used in the family, and on the farm, and wasted. There remains 1,000 bushels. As a portion of this may be represented by less bulky and more valuable articles, we again reduce it one-third, to get at the weight that will require transportation to a market. This leaves us 666⅔ bushels of corn, or an equivalent in other things, and which, multiplied by the number of families, gives equal to 33,333,000 bushels of Indian corn. The usual mode is to estimate 33 bushels to the ton. At 33⅓ it gives 1,000,000 tons. The distance which it would have to be transported to market could not be less than 100 miles, and the price would be low at five cents per ton per mile. This would give $4,000,000!'

"Making the application with the substitution of 22 inhabitants to the square mile, instead of 15 (which, under the Census of 1850, we find to the square mile along the line of the Alton and Springfield Railroad,) and we have the following result: Length of Road, 72 miles. Number of square miles within 15 miles of the Road, 2,160; population 50,160,—farmers engaged in cultivation of the soil, 8,360; corn produced, 12,540,000 bushels; estimated amount of corn, or its equivalent, to pass on the Road, 5,573,333 bushels;—or, 666⅔ to the

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family, and amounting to 167,535 tons; which, at four cents per ton per mile, averaging the transportation at one half the length of the road, 38 miles, at $1 52 per ton, will yield a revenue of $254,653 20—more than double our estimate.

"3d. The third item is the return of freight, exclusive of coal and lumber; in relation to which Mr. Neal remarks:

"'The returns that will be made to the producer of this large amount of property must bear some proportion to it in value and in bulk. If the corn nets but fifteen cents per bushel, it will give to each family $100; and to the 30,000 families, $5,000,000. To keep within bounds we will suppose that, exclusive of the lumber by the coal and other trains, only one-eighth of the outward tonnage is returned in supplies. This would give the Road, at the enhanced rate which such goods would bear, (say five cents per ton per mile) or $5 per ton, $625,000. Applying a similar estimate to the Alton and Sangamon Railroad, and we should have 20,941 tons, at five cents per mile per ton, averaging to be carried 38 miles; which, at $1 90 per ton, would give a revenue of $39,787 90.'

"4th. The 4th item is local travel; concerning which the following language is used:

"'The local travel of the same population will be an item of some importance. They must be considerably scattered, and if they associate at all, they will use the Road. Each head of a family averages five journeys of 60 miles each, or 300 miles per annum, for the whole household. It will cause the Road to carry one passenger 18,000,000 miles; which, at three cents per mile is $540,000. This would amount to nine dollars for each family per annum. If we reduce it one half, there can be little doubt of the other half being made up, and more than made up, by the other local passenger and freight business, such as those living without the line of twelve miles, of citizens of other States visiting the stations, and the thousand occasions for trips from the termini to the interior.' It seems safe then to let this item stand. Applying this estimate to the 8,360 families on the line of the Alton and Sangamon Railroad, and we have a revenue of $75,240 from this source.

"5th. The fifth item has reference to the through travel, which must depend upon so many conditions differing from those which belong to the Central Road, that we have thought it best to retain our own estimates of thirty through passengers each way per day, which, at three cents per mile, will give $53,640. Mail, 72 miles, $7,200; and we have the following

"RECAPITULATION:

"Freight of coal and lumber.....	$22,800 00	
Freight of produce of the soil.....	254,653 20	
Freight returning.....	39,787 90	
Local travel.....	75,240 00	
Through "	53,640 00	
Mail	7,200 00	
	$453,321 10	
Less expenses, 50 per cent.....	226,660 55	
		$226,660 55

—over 22 per cent. on the cost of the Road.		Mr. Hammond.
Or if we take the above	$453,721 10	
And deduct all the items not strictly calculated according to Mr. Neal's basis, to wit: items 1 and 5	76,440 00	
We have still left	$377,281 10	
Less expenses, 50 per cent	188,640 55	
Leaving over 18 per cent. profit	$188,640 55	

"We have been particular to give Mr. Neal's estimates, for the reason that he is connected with the greatest Railroad project now occupying the attention of our people, and exerts a strong influence in the direction of its affairs. His estimates are undoubtedly prepared with the greatest caution, and upon the most careful investigation, and deserve additional consideration from his justly celebrated reputation, of being one of our ablest and most successful Railroad men. He gives them as his own conviction, of what will be the results of the Central Railroad, upon a given state of facts. If they will prove true of the Central, they will equally so, of most of our Western Roads. They are certainly applicable in their fullest extent to the Alton and Sangamon Railroad, as the country traversed by this is richer, more densely populated, produces more to the acre, and is probably more fertile than the country traversed by the Central Road.

"We have thus far spoken only of the local business of the Alton and Sangamon Railroad. We may here add one or two general remarks in reference to the comparative amount of freights upon Eastern and Western roads:—1st. That all articles of Western export are of a bulky kind, and pay a large profit in proportion to their value. 2nd. That agriculture, being the leading pursuit in that quarter, all surplus raised is exported to distant markets, and must pass over Railroads when constructed; that for the want of manufacturing establishments, nearly every article of consumption that the farm does not produce, is imported from abroad, or from the other States. This state of things necessarily causes a much larger amount of transportation, than in the older States, where the greater variety of pursuits, enables every person to supply most of his wants from the industry of his own neighborhood. The necessary interchange there is effected without going beyond the circuit of a few miles.

"What will be the through travel and business of this road? We may set it down as a fixed fact that the *Central* Road will be completed at an early day. Bloomington, a town about sixty miles northwest from Springfield, is to be a point in this Road. From the latter place, active measures are in progress for the construction of a Railroad to the former. This road will undoubtedly be in readiness as soon as the Central Road is opened to the lake. A line drawn from Chicago to St. Louis, would very nearly pass through the three towns of Alton, Springfield and Bloomington. A road through these intermediate places will constitute the shortest practicable line between the two great Cities of Illinois and Missouri. Travelers from the north,

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wishing to reach St. Louis on the Mississippi, would leave the Central Road at Bloomington. The Alton and Sangamon Railroad must, therefore, for aught we can see, forever constitute the lower and most profitable part of the *main trunk line,* between Lake Michigan and Springfield, Alton and St. Louis. The latter, we believe, is the great city of the West. In a commercial point of view, the south shore of Lake Michigan is one of the most important points in the United States. It is the ultimate of Lake navigation, and around it sweep all our northern lines of Railroad, running east and west. The through travel must equal, if it does not greatly exceed, the local. We think that no one who will examine a map of Illinois, with the lines of Railroad in progress, can fail to come to similar conclusions.

"The same general conclusions apply with equal force to the transportation of freight. In a good stage of water, the majority of the heavy freight will follow the Canal and the Illinois river. But even at such periods the more valuable and perishable articles will go by Railroad. The same causes which constitute the above the route of travel will make it the route for freight. When the river is not navigable, from low water or ice, the amount of freight thrown upon this road must be very large. A great part of the West will receive their foreign supplies through Chicago, from the low cost of carriage between that city and New York. These goods will be distributed over the country chiefly through Railroad; and from the certainty and speed of transportation, St. Louis and Alton will at all times receive no small portion of their merchandise over the above Road.

"We are happy to assure the Stockholders, and the public, that the Road is now making good progress toward Springfield: a large and efficient force is employed at all important points along the line, and it is believed that the whole road will be ready for the rails before the first of May, 1852. Eleven miles of the track is laid, and the iron for the whole road has been purchased, and thirty-three miles of it delivered in Alton, and the balance in New Orleans; 50,000 Cedar cross-ties have also been delivered at Alton, and the balance required for the whole Road are on their way. The Engines and Cars have all been contracted for from the best builders, and a portion of them delivered. The state of the graduation is such that the laying down of the track will be continued without interruption, and in the shortest time, to its entire completion.

"The Directors, influenced by the consideration that the results indicated in the foregoing remarks, will be realized in the completion of their Railroad as a mere local enterprise, and looking to the great accession to their business which may reasonably be expected from the further extension of their Railroad, by a resolution, dated May 30, 1851, directed their Engineer to proceed 'forthwith to make an examination and survey of the Extension of their Railroad from Springfield to Bloomington, with a Branch to Pekin and Peoria.'

* * * * * * *

"The estimates are predicated on a road-way in excavation of twenty feet, and on embankment of fifteen feet width, with slopes of one and a half to one.

"COMPARATIVE TABLE

Mr. Hammond.

Of Distance and Time, by River and Railroad, taking the Rivers at their best stage, and rate of travel on Railroad, at 25 miles the hour, allowing two hours time, and 20 miles distance, for the Ferry between ST. LOUIS *and* ALTON.

NAMES OF PLACES.	DISTANCE. [MILES.]		TIME. [HOURS.]		DIFFERENCE.	
From—St. Louis to	River.	R. Road.	River.	R. Road.	In distance.	In time.
Burlington	230	203	30	10	27	20
Oquawka	247	200	30	10	47	22
Rock Island	318	301	40	14	17	26
Galena	408	328	50	15	80	35
Peoria	203	157	26	8	46	18
La Salle	267	211	34	10½	56	23½
Springfield, by River and S. & M. R. R.	161	92	24	5	69	19
Jacksonville	128	125	20	9	3	11

* * * * * * *

"The estimates for bridges are based on the cost of an iron bridge, on the plan of that across the Canal Basin at Albany, built by S. Whipple, Esq., of Utica. The great scarcity, and consequent cost of timber for such structures, renders it more particularly desirable that Railroads should dispense with the use of timber, as far as practicable, in the West; and my opinion is, that a proper regard to the true interest of Railway proprietors, should induce Engineers to urge this point.

"I am aware that there is a prejudice in the minds of many distinguished Engineers against the use of iron for bridges, owing to a supposed uncertainty which is stated to exist, as to the strength of the material used. Experience has shown that no subject, in the range of Civil Engineering, has so much exercised the ingenuity and so often disappointed the expectation of the Engineer, as the construction and maintenance of Bridges. Indeed, I believe it may be said of Railroads generally, that all their bridges have been in the first instance too slightly built. Universally on the Pennsylvania works, the older plans have been abandoned to give place to new structures; and on the New York and Erie Railroad, the same process is now going on. Nor are these works exceptions. These structures, for the most part, are of wood, and in view of the above facts, I feel bound to say that I can perceive no objection growing out of the use of iron for bridges, which is not equally applicable to the use of timber. Suitable tests are in use by which the strength of iron may be fully and certainly ascertained; and it cannot be denied, that in durability, accidents from fire, and security from malicious designs, they are far superior to any structure of wood which can be devised.

"With respect to this particular enterprise, I believe that a permanent, indestructible iron bridge may be obtained at a cost not exceeding that of a wooden structure, of the most improved construction. I have accordingly provided in the estimates for iron bridges.

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"The scarcity of good timber for the superstructure, and the cost of Oak ties of suitable quality is such, that I have estimated for a superstructure with Cedar cross-ties, ten inches broad, six inches thick, and eight feet long.

"The iron is of the H form, 56 pounds to the yard linear, laid upon ties 30 inches from centre to centre, without juncture plates.

"The depots, engine houses, car houses and shops, water stations, turn-tables, &c., are similar to those on the Michigan Central, and to compare in style with those already erected on this Road.

ESTIMATE:

Graduation and masonry, including bridges, grading of side-tracks, depots, &c., complete from Springfield to Bloomington		$310,567 75
19,200 rods fencing, at 75 cents per yard		14,400 00
SUPERSTRUCTURE.		
2,112 cross-ties of Cedar, at 60 cts	$1,267 20	
88 tons Iron, at $50 per ton,	4,400 00	
4,500 lbs. Spikes, at 6 cents,	270 00	
Workmanship, per mile,	500 00	
62 miles track, including sideings, at	$6,437 20	$399,106 40
Engine, car, and depot houses, and fixtures, at Bloomington		15,000 00
Machine shop and intermediate water stations, &c		10,000 00
Engineering and contingencies		50,925 85
59 32-100 miles at $13,486 18 per mile, is		800,000 00
Graduation, masonry and bridging, grading depots and side-tracks, of Pekin and Peoria branch	$321,942 72	
Fencing 12,800 rods, at 75 cents	9,600 00	
SUPERSTRUCTURE.		
42 miles track, including sideings, at $6,437 20 per mile, is	270,362 40	
Engine, car, and depot houses and fixtures, at Pekin, and Peoria, and intermediate stations	15,000 00	
		616,905 12
Engineering and contingencies		33,094 88
		$1,450,000 00

Furniture and equipments, will be required, as follows:

5 Engines	at $7,500	$36,500 00	
5 Passenger Cars	at 2,000; 56 seats,	10,000 00	
25 Covered "	at 750	18,750 00	
50 Open Freight Cars,	650	37,500 00	
50 Cattle and Hog Cars,	650	37,500 00	
			140,250 00
Total			$1,590,250 00

"I have made no estimate of the cost of right of way. In most instances, where the land is under cultivation, the right of way will be given if the fencing is provided for, which is an item I have estimated. Where it is uninclosed the present price per acre will range from five to ten dollars. Mr. Hammond.

"In reference to the general results shown in the above estimate, I remark that the average cost per mile a little exceeds the work on the line between Alton and Springfield, owing to the crossing of Salt creek, Sangamon and Mackinaw rivers; but the aggregate, when compared with any hundred miles of Railroad line in the East, indicates very strongly the fact, that while the resources of the country very far exceed any portions of the North and East, the cost of construction is at least sixty per cent. less.

"In conclusion, I have thought it would be desirable to place before our Stockholders, a brief view of the various roads under construction or in contemplation, which will exercise an influence on the prosperity of the State, and necessarily upon that of our Roads. So far as competition may or does exist, I consider it salutary and tending to the prosperity of all the different corporations, who will be impelled by it to do better service to the public, and thus secure more permanently their own interests, than they otherwise would be induced to do. And it is certain that such rivalship though salutary, must exert itself more in appearance than in fact, for the natural increase of business and wealth growing out of the construction and operation of these Roads, together with the increase of population, will, by adding to the general prosperity, benefit the whole.

"On reference to the outline map herewith, it will be perceived that at Springfield we connect with the Sangamon and Morgan Railroad, which has for several years been in active and profitable operation, a distance of fifty-seven miles. In addition to the local business from Springfield, I think we may look for a large business from the thriving farmers along the line of that Road, as well as from the beautiful and populous village of Jacksonville, and its vicinity. The surplus products of the country are determined north or south as the market rules. For a southern market they would find it for their interest to send over our Road to Alton and St. Louis, and for a northern market by our Road to Bloomington, and thence by Central Railroad and Rock Island Railroad to Chicago; freights in return would take the same course, and we should contribute as much to the business of that Road, if not more, than they would obtain otherwise. At the western end of the Sangamon and Morgan Road, the extension towards Quincy is under contract for some thirty miles; and its early completion may be expected. Eastward, from Springfield, following the old line of the Northern Cross Railroad, arrangements are in progress to enable a connexion to be made with the Central Railroad at some convenient point. I feel quite certain that this design will be carried out; and I doubt not we shall receive a large addition to our business from it. It seems to me, that so far as the course of business will be affected by the opening of our Road to Springfield, that it will reverse the direction of the business on the Sangamon and Morgan Railroad, so that the balance of the business will tend towards Spring-

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field, instead of the Illinois. Among other reasons for this belief, it may be remembered that the time from Springfield to St. Louis is now by Railroad and steamboat twenty-four hours; on the opening of our Road it will not exceed five hours. If then the Sangamon and Morgan Railroad should carry their passengers for nothing, they would still have nineteen hours time against them. In reference to freights also, the prices from Springfield will not exceed the river freights from Naples; shippers will therefore prefer to ship to Springfield rather than to Naples, saving thereby time and insurance. Under these circumstances it is not too much to expect that the business of the country, so far as these two Roads are concerned, will be conducted on fair and equitable conditions. The value of this connexion, by which the lime, coals, lumber and merchandise from the south, may be distributed for sixty miles on each side, is certainly very great.

"From Springfield to Bloomington, the local business will be sufficient to maintain the Road very shortly. The distance from Alton to Galena by this route is 307.68 miles; or in time, thirteen hours. At La Salle, the connexion with the Rock Island Road, extending to Rock Island on the one hand, and Chicago on the other, to the Central Michigan Road, shows how nearly we occupy a position on the great main line of transit.

"Following our line to Pekin and Peoria, we find the only crossing on the Illinois by a bridge; and having crossed it, we unite with the Peoria, Burlington and Oquawka Railroad, through the center of the Military Tract, one of the richest and most thickly settled portions of Illinois. In addition to this, following the line of the Illinois river northward, we command the business on both sides of this river for sixty miles. I hesitate not to say this, because we are nearer to St. Louis by forty-three miles in distance and fifteen hours in time, than by the river in the best stage. The average charge for freight will be less than steamboat rates, one season with another. This is a matter which is susceptible of demonstration. Omitting, therefore, all the lines which may be said indirectly to connect with us, we may consider the connexions formed by our extended line as equal to six hundred and sixty-three miles. If, then, the reasoning in the prefatory remarks to this report are reliable, every foot of this extended line will be fully and profitably engaged, and the main branches, where the trade will concentrate, will be compelled to make arrangements for the transaction of a business unparalleled for its importance and extent in the history of commerce.

"The present population of the Counties traversed and touched by the Alton and Sangamon Railroad, and extensions to Bloomington and Peoria, is one hundred and forty thousand; and when it is considered that St. Louis, with her eighty thousand inhabitants, and upwards, is only separated from one end of the Road by a short ferry, and that the Peoria and Oquawka Railroad is an extension of the Peoria Branch, it will be seen that a larger population are directly reached and accommodated than by any Road of the same length in the West.

"The talented and efficient Editors of the Western Journal, and of other papers in St. Louis, have been urging on the capitalists and

merchants of that city, the imperative necessity which exists, that every exertion should be made to control the business which seems likely before long to leave its usual course, and flow in another direction. The clearing out of the Rapids on the one hand, and the Shoals at the mouth of the Mississippi on the other, is both useful and necessary; but the influence St. Louis should exert, is to be displayed by opening the most direct, cheap, and speedy routes to their great centre of business and population. How then can this be done with Peoria, Burlington, La Salle, Rock Island, Galena and Chicago? To my mind there is but one answer, and that is to unite in the construction of this line from Springfield north. Mr. Hammon

"The time required to complete these lines would not exceed two years; and if ample means were provided they might be put in operation sooner. The most careful examination seems only to add force to the conviction, that there is no project now before the public, which will so well remunerate those who may invest their capital in the stock, or from which a larger dividend may be expected.

"I take pleasure in acknowledging my obligations to P. A. Badeau, Principal Assistant, and R. R. Powell and T. Connelly, Junior Assistants, for the efficiency of their services in the field and office.

"Respectfully submitted,

"J. I. SHIPMAN,
"Chief Engineer.

"*To the* President and Directors
of the Alton and Sangamon Railroad Company,
"Alton, *January*, 1852."

The map, referred to by Mr. Shipman, and annexed to his report, shows the railroads built and building in Illinois in 1851, and includes the entire State of Illinois, which may not improperly be termed the heart of the American railroad world, as we know it. The map shows that only the following railroads were built and building at that date:

Galena and Chicago Railroad, 183 miles;
Rock Island Railroad—Rock Island to La Salle—80 miles;
La Salle to Chicago, mileage not given;
Burlington and Peoria Railroad, 92 miles;
Sangamon and Morgan Railroad, from Quincy on the Mississippi River to Meredosia, 60 miles; and thence to Springfield, 55 miles;—115 miles in all. (Of this road, Mr. Shipman was also Chief Engineer.)
Alton and Sangamon Railroad, from Alton to Springfield, 72 miles.

Numerous projected railroads are shown, and the "National Road," crossing the State from Terre Haute to Alton, the famous old wagon road of the pioneers, over which passed the "prairie schooners" from the east to the California gold fields in 1849, which migration was still at its height when Mr. Shipman wrote his report.

Mr. Hammond. The map also shows a small portion of the Michigan Central Railroad, entering Chicago, and a portion of the Hannibal and St. Joseph Railroad, leading out of Hannibal. Only one railroad is shown leading out of St. Louis, and this is named the Pacific Railroad.

Mr. Shipman returned east in 1855, leaving his assistant, Mr. Floyd-Jones, to become Chief Engineer in his place on the construction of the railroad from Bloomington toward Chicago. After his return east, he again became Chief Engineer of the Long Island Railroad, and, as such, constructed nearly all its numerous branches. He died in 1884. In his youth he began his professional career about the year 1834 under Major W. G. McNeil (whom he succeeded as Chief Engineer) on the Brooklyn and Jamaica portion of the Long Island Railroad, and during this early period, the writer is informed, the late James Pugh Kirkwood, M. Am. Soc. C. E., afterward President of this Society, was his assistant. He retired from active professional work several years previous to his death, mainly in consequence of almost complete loss of hearing.

The writer desires to obtain all possible data relative to Mr. Shipman, and will be greatly obliged for corrections and additions to the foregoing.

Mr. Wood. Fred. J. Wood, M. Am. Soc. C. E. (by letter).—Massachusetts, like the other twelve States, at the close of the Revolution, felt the effects of British restrictions on inter-colonial trade, in having no adequate interior communication; and the movement for better means of transportation began almost coincidently with the establishment of our new government. Unlike New York, Boston was not at the mouth of a mighty river, penetrating well into the interior and bringing the primitive commerce to her wharves, and artificial roadways were required to enable her to make even a pretense of competition, as all the natural routes of commerce led from, and not to, the Massachusetts capital. The opening of the Middlesex Canal, connecting Boston with the Merrimac River at Lowell, gave a route for the produce of New Hampshire and parts of Maine, Vermont, and Canada, preferable to the route by the Connecticut River to New York, and the commerce, thus diverted, together with that brought in over the network of turnpikes, constructed during the years following 1795, gave Boston a prestige not insignificant beside that of New York. In 1825, however, when the vast Lake region became tributary to New York by the opening of the Erie Canal, Boston, like many other coast cities, faced the danger of being utterly eclipsed by the growth of her rival on the Hudson. Many extravagant propositions for canals, by which the trade of the growing West could be brought to Boston, were exploited, one going so far as to propose a tunnel where the Hoosac Tunnel now serves the desired purpose, but all failed to secure the

needed support, and Boston's failing courage was only renewed by the rumors of the success attending the operation of the new railways in England. The courage thus revived, however, was not sufficient to consider the undertaking of such works by private means, and the earlier agitation was all for railways to be built by the State. Mr. Wood.

On February 22d, 1826, a resolution* was offered in the Massachusetts House of Representatives, providing for the appointment of a commission to inquire into the practicability of constructing a railway from Boston, and make such further investigations as would enable the Legislature to judge the advisability of commencing such work; but the time was not ripe, and the resolution was laid on the table, there to lie while the private energy of Bryant and Perkins built, and demonstrated the success of, the Granite Railway, in Quincy, Mass. The success of that road, 4 miles long, and built between April 1st and October 7th, 1826, was the cause of a petition being presented to the Massachusetts Legislature late in 1826,* asking for railway surveys. Then a commission was appointed,* as outlined in the above-mentioned resolution, and, in consequence of its favorable report and of further agitation by leading men of Boston, an appropriation of $10 000 was made* for investigations which were carried on during 1827; and, in 1828, a Board of Internal Improvement of nine members was appointed to investigate thoroughly the propostion of railways from Boston to Providence, and to Albany.

The report of this board, submitted to the Legislature in January, 1829,† contained detailed reports, with descriptions and estimates, by the engineers, James F. Baldwin and James Hayward, with maps and profiles of several routes surveyed. The report, after describing the methods of track construction used on English roads, concludes that, owing to the high price of iron in this country, and the plentiful supply and cheapness of suitable stone, a better method would be to construct a continuous stone wall under each rail, laid deep enough to be below the action of frost, covered by a rail of split granite 1 ft. wide and 1 ft. deep, hammered on top and on the upper edge of the inner face, with a bar of iron for the wheel to run on, fastened to the stone at each foot by bolts or rivets, these rails to be placed at a distance of 5 ft., and the space between them to be filled within 6 in. of the upper surface with earth and gravel to form a path for the horses.

It will be noticed that locomotives had not then been considered, all investigations having been along the line of horse-drawn cars or trains. The report was supplemented by one from Solomon Willard, architect of Bunker Hill Monument, who traveled over much of the

*Massachusetts Archives.

†Report of the Board of Directors of Internal Improvements of the State of Massachusetts on the Practicability and Expediency of a Railroad from Boston to the Hudson River, and from Boston to Providence, 1829.

Mr. Wood. State, examining the suitability of various supplies of stone and convenience of access, and who made the following estimate of the cost of four rails:

"For quarrying the rail stone.....	5 cts. per foot.
Dressing and preparing for the work....	4 cts. per foot.
Hauling to the line of the road.....	4 cts. per foot.
Opening quarries and making roads.....	1 ct. per foot.
Making for these items, 14 cts. per foot =	$9.24 per rod.
Digging, hauling, laying the trench wall..	3.84 per rod.
Excavating four trenches.....	0.50 per rod.
Laying rail stone and putting on iron...	2.00 per rod.
Transporting iron rails.....	0.50 per rod.
	$16.08 per rod."

The Quincy road was originally built* with stone ties, spaced 8 ft. apart, and wooden rails faced with iron, but the wooden rails quickly decayed and were replaced with stone.

The report of the Board was a masterful treatise on a subject of which so little was known, and went at length into consideration of the increased tractive power of a horse on such a road, gave estimates of expense of operation, statistics of past travel and freight, and consideration of the question of how to apply additional power for surmounting steeper grades. For grades not exceeding 26 ft. per mile, the load for a single horse is given as 10 tons; with 80 ft. per mile as the steepest, 5 tons per horse. The cost of transporting a ton of merchandise from Boston to Albany was computed at $2.97, for which 4 days would be necessary. Passengers were to be carried in 22 hours for $3.05.

Boston, between 1820 and 1825, gained 15 000 in population, or more than 30%; from 1825 to 1830, the period now being considered, the gain fell to only about 3 000, or about 5 per cent.† In 1829, therefore, the population was about 60 000, and the merchants were seriously alarmed over the loss of trade felt throughout the city. A general meeting of the inhabitants was held on February 10th, 1829,‡ at which, following speeches which deplored the falling off in the trade of the city, resolutions were adopted endorsing the railway projects by the State, but calling for private authorization if the State did not undertake the work. On the other hand, the little Town of Hawley,§ out in Franklin County, filed its remonstrance at the State House on May 25th, 1829, against any form of railway promotion. A railway,

*Stuart's "Lives and Works of the Civil and Military Engineers of America."

†"Boston's Growth. Published by the State Street Trust Company.

‡Broadside—"The Great Railway Meeting"—Boston Public Library.

§Massachusetts Archives.

said Hawley, would do it no good, but, by reducing the price of commodities, would injure it. The estimates did not show one-half of the cost, and the resulting revenue would not be sufficient to pay operating expenses. Various citizens of Foxborough* joined in a petition to the Board, stating that, in their opinion, a better route than that surveyed could be found for the railway to Providence, by following the line as set forth in the petition, which, by the way, would take away the possibility of the village being disturbed by passing cars. The railroad of to-day little resembles the kind proposed in 1829, but the same old opposition prevails. Mr. Wood.

The Legislature of 1829, however, seems to have been of the hard-headed, practical kind, which, taking little stock in theoretical reports and "surveyors' pictures," preferred to judge the subject by hard-and-fast facts. Although Massachusetts was the pioneer in railway building, by having the Granite Railway in operation in 1826, other States had outstripped her, and, in 1829, extensive improvements were under way in New York, Pennsylvania, Maryland, and South Carolina. So a committee, consisting of Edward H. Robbins and James Hayward, the latter being the engineer who had surveyed the routes to Providence, was appointed† to visit the "works of internal improvement" in Pennsylvania and Maryland, and to return with accurate information of what had actually been done and what it was really costing. This committee first visited the works of the Delaware and Hudson Canal and Railroad Company, traveling by canal from Kingston, on the Hudson, southwesterly to the Delaware River near the northern extremity of New Jersey, up the Delaware to the Lackawaxen, and along the Lackawaxen to its head near Honesdale, a distance of 108 miles. The committee found that the canal cost, on an average, about $18 000 per mile, including much rockwork, and that the lockage was 1 088 ft. From Honesdale the committee went to Carbondale by rail, 16 miles over Lackawannock Mountain, requiring a rise of 800 ft. in the first 3½ miles. This was surmounted by inclined planes of 1:12 and 1:20 inclinations, up and down which cars were passed by stationary engines and endless chains. Beyond the summit, cars were operated by gravity or horse power. The committee rode up a plane 3 or 4 miles in length, in a train of four "waggons (a net ton each)" in which were twelve men "with some other luggage," all of which was drawn by "a small horse of very ordinary appearance which did not break his trot for the whole distance." The track was composed of 6 by 10-in. wooden rails, with two ½ by ½-in. iron plates on them, and supported on cross-sleepers at intervals of 10 ft. The committee states that it is a "well ascertained fact that a horse will draw a much greater load upon

*Original paper in possession of the writer.
†Massachusetts Archives.

Mr. Wood. a railway of granite than on one of wood." More than 400 tons a day had been passed over this road, the cost of which, exclusive of engines and apparatus for passing the planes, had been $6 500 per mile.

From Carbondale the committee went down the Lackawannock River to the Susquehanna, inspected the work on the Middle Division of the Pennsylvania State Canals, and gathered much information relative to the projected system of canals by which the State was to be covered. In the Susquehanna Valley the Pennsylvania Railroad was under construction by the inhabitants of the valley, who deemed it best to connect the Pennsylvania canals with Philadelphia. An easy route would have been to follow the river to Baltimore, but the railroad gave trade to Philadelphia, and also gave them two markets for their goods. The work of constructing the Baltimore and Ohio Railroad was then inspected by the Massachusetts men, who found 2 or 3 miles actually completed and 25 miles more ready for the rails. In its report, the committee dwelt at length on the costly features of the work in Maryland, which would not be met in Massachusetts, and it is evident that the opponents of railways had attacked the estimates submitted for Massachusetts construction, using more or less accurate data from the Baltimore and Ohio for their weapons. The report states that the original estimates had been overrun, largely on account of substituting stone bridges for wooden, incurring a cost of $336 000 for this item in the first 25 miles; and triumphantly points to the fact that the costs ascertained on the railway and canal work in Pennsylvania confirmed the Massachusetts estimates.

Concluding, the committee expressed the conviction that railroads are far better than canals, and that Massachusetts is as well adapted for their construction as any place, and hoped the State would not fall behind the others in such work.

To clinch still further the grip on hard-and-fast facts, Governor Lincoln, of Massachusetts, addressed a letter to the President of the Baltimore and Ohio Railroad Company, requesting specific information on the following points:

1.—The progress already made on the Baltimore and Ohio road.
2.—The manner in which the work is constructing.
3.—Cost of grading the way, etc., etc.

This letter was answered at length by Colonel Long, President of the Board of Engineers of the road, under date of December 1st, 1829.* Of the progress he reports a "definitive location" from Baltimore, 70 miles to the Upper Point of Rocks on the Potomac River, with a minimum curve radius of 400 ft., with three exceptions of 318, 280, and 210 ft., respectively. The grades were carefully considered in relation

*Massachusetts Archives.

to the tractive power of a horse at different speeds, etc., and assuming that 5 tons would move eastward to 1 ton westward, it was concluded that a descending grade toward Baltimore of 30 ft. per mile was as good as a level, while one of 15 ft. was better; but a limit was placed at 26 ft. per mile because, in bad weather, the wheels would slip on the rails, making it necessary to install stationary brakes if steeper declivities were allowed. Twenty-five miles had been graded and stone bridges built, involving 660 000 cu. yd. of excavation, 630 000 cu. yd. of embankment, and 25 000 perches of stonework, at an average cost of $58 000 per mile. A unit price of about 28 cents per cu. yd. is shown by the cost of the excavation. The topography is described as very rough, making expensive construction necessary, but, from the Forks of the Patapsco, 25 miles from Baltimore, to Ellicott's Mills, the line followed the face of the hills more, and the cost per mile fell to $38 500, while, on the second division, extending from Ellicott's Mills about 12 miles westward, the cost was expected to be only $10 300 per mile. It was then proposed to pass the summit between the Patapsco and the Monocacy by inclined planes, each 1 mile in length and ascending at an inclination of 2½°, and operated by stationary engines. Mr. Wood.

Colonel Long's description of the manner of constructing the track is especially interesting, and noteworthy on account of the attention given to drainage. The tops of fills and bases of cuts were alike made 26 ft. wide, with side slopes of 1½ : 1 for fills, and 1 : 1 for cuts. Of the 26 ft., 20 were macadamized with broken stone of 2 and 2½-in. size, laid to a depth of 4 in. Trenches were dug through this pavement, 4 ft. apart, to receive the ties and at each end a pit was dug, 18 in. long, 12 in. wide, and 12 in. deep, which was filled with rubble to form a foundation for the tie. The ties, of locust or cedar, were 8 ft. long and 7 in. in smaller diameter, and were notched to receive the wooden rails, the outer edge of the notches being placed to a true spacing of 5 ft. The wooden rails were of 6 by 6-in. Southern heart pine, in lengths of from 15 to 40 ft., and were set in the notches with keys. The iron rails, on which the wheels were to run, were of wrought iron, $\frac{5}{8}$ by 2¼ in. and 15 ft. long, "appropriately rounded on their upper sides and perforated with elliptical holes about 15 inches asunder." At the joints they were scarfed on an angle of 60° with the sides, and laid on a plate $\frac{3}{16}$ in. thick. The nail holes were countersunk, allowing the nail head to be driven below the touch of the wheel, while the elliptical shape of the hole took care of expansion and contraction. For a width of 9 in. on the inner side of each wooden rail, coarse broken stone was laid, leaving a space of 2½ ft. in the center which was filled with finer broken stone to form a path for the horses. An alternative construction of stone rails was contemplated, in which case the stone rails were to rest on continuous rubble walls built in trenches to a depth

Mr. Wood. below frost. Wooden rails were adopted for first construction, because it was believed that the fills would not sufficiently compact to receive such permanent work as stone, for 4 or 5 years. From figures given, it is seen that the material for the track cost per foot as follows:

One-quarter of a tie, at \$0.35 + \$0.08 for notching and dressing	\$0.108
Wooden rails, 6 by 6 in., at \$0.06 per running foot	0.12
Nails, at \$0.09 per lb	0.023
Iron rails, at \$58 per ton delivered	0.272
	\$0.523

The cost of grading is given as ranging from 12½ to 50 cents per yd., averaging 28 cents, as already stated. Overhaul was found to double the cost at a quarter of a mile, and to treble it at a half mile. Blasting cost from 50 to 87½ cents per yd.

The various reports received seem to have settled forever the superiority of railroads over other means of communication, and, while satisfying the State Government, satisfied private investors as well, for it is found that three corporations were chartered to build commercial railroads, in March, 1830. Although no roads were built under those charters, the precedent was established, and, commencing in 1831, a long list of successful commercial railroads was chartered, and the transportation development of Massachusetts has since been in private hands.

The early charters for railroads were framed on the supposition that they would be used like turnpikes, that for the Boston and Providence, granted in March, 1830, containing the following:*

"Section 6. Be it further enacted, That the directors of said Corporation, for the time being, are hereby authorized to erect toll houses, establish gates, appoint toll-gatherers, and demand toll upon the road when completed, etc., etc."

Availing themselves of this privilege, certain parties, in 1837, secured a charter for the Seekonk Branch Railroad, intending to build only ¼ mile of railroad of their own at the Providence end, and a station of their own in Boston and to use the whole intermediate track of the Boston and Providence Railroad Corporation, over which to run their independent engines and trains, paying toll for the same.† The operations of this company were a serious annoyance to the management of the Boston and Providence for about 3 years, until the latter company bought out the Seekonk Branch company and

*Special Laws of Massachusetts, 1830.

†Article on Railroad History, by Edward Appleton, Railroad Commissioner of Massachusetts, 1869-70, published in The Atlas of Massachusetts, 1872.

secured the enactment of a law by which one railroad company could not enter on the tracks of another without permission. Mr. Wood.

The construction of the early roadbeds forms an interesting section of railroad development. In the report to the State Legislature in February, 1838, the Boston and Lowell gives the following:*

"Manner in which the rails are supported. The foundation for the first track of rails is laid with dry stone walls on trenches from 2½ to 4 feet deep, and about 18 inches thick. The rails are laid, part on stone blocks, part on stone blocks and sleepers, and a small part on stone blocks and wooden sleepers, all of which are supported on the trench walls above described. The stone blocks are 3 to 4 cubic feet each. The stone sleepers are 7 feet long and average 8 x 10 square. The wooden sleepers of chestnut and white cedar are 7 feet long, 7 to 8 inches diameter. This track is mostly laid with rails of the fish belly pattern, and is set on chairs which are fastened on to the blocks and sleepers above described."

A section of this track, rolled in Liverpool in 1834, laid on the Boston and Lowell in 1835, and taken out of the Lowell yard in 1897, now ornaments the corridor of the offices of the Massachusetts Board of Railroad Commissioners. Such track lasted a much shorter time than its substantial construction merited, for we find, in 1845, the same company reports:†

"The road has a single track with a heavy T-rail of 56 lbs. to the yard, upon chestnut sleepers of 7 feet long and 6 inches in depth, 2 feet 7 inches apart, resting upon a bed of clear gravel at least 2 feet deep. The rails are in lengths of 18 feet, and the joints are secured by a clasp chair of 20 lbs. weight."

The Boston and Providence track, in 1837, is thus described:‡

"The rails on the main road are edge rails of the T-pattern, and are supported on white cedar sleepers, laid three feet apart from center to center. The greatest part of them are laid directly upon the earth; in some few cases on longitudinal sills, and, in a few cases, on walls built of small stone. The rails on the said branch" (Dedham) "are plate rails, fastened to wooden sills of southern pine, six inches square, the whole being supported on white cedar sleepers, laid three feet apart from center to center, and placed directly upon the earth."

After eleven years of operation, this road reported§ that it had been necessary to replace only 750 rails, or 2¼% of the whole. The average life of the white cedar sleepers had been from 7 to 8 years.

The Boston and Worcester, in February, 1838, reports:‖

"The iron rails, which are parallel edge rails, are supported by cast-iron chairs, on cross-sleepers of wood, mostly cedar and chestnut. The

*Massachusetts Railroad Reports for 1837.
†Massachusetts Railroad Reports for 1844.
‡Massachusetts Railroad Reports for 1837.
§Massachusetts Railroad Reports for 1844.
‖Massachusetts Railroad Reports for 1837.

Mr. Wood. ends of the sleepers rest on a bed of stone rubble, placed under the rails, along the whole length of the road."

The Western Railroad, in 1841, thus describes its track:*

"The edge rail of the T-pattern is used, weighing 56½ lbs. to the yard, supported by wooden sleepers, 7 feet long, and 7 inches through, at distances of 3 feet from center to center. These sleepers rest on longitudinal sills of plank, 3 inches thick, and 8 inches wide, with a short piece, 3 feet long, under them at their joints, and four additional pieces, three feet long under the sleepers at the joints of the iron rails."

The Boston and Maine, in 1844:†

"The earth excavations, and embankments are levelled off, and one and a half feet of sand, or gravel, is then filled on to the road; the sub-sills of plank are then laid longitudinally, and the sleepers of chestnut, cedar, or hackmetac, are laid transversely, partly two and one-half feet, and partly three feet apart. Iron rails of the T-pattern are then laid, supported at the joints by cast-iron chairs, and spiked to the sleepers; sand or gravel is then filled in between the sleepers."

The writer has looked eagerly for cost data on the early work without very satisfactory results. The following items, although fragmentary, may prove of interest:

At the opening of the year 1839, the Eastern Railroad was in operation from Boston to Salem, a distance of about 13 miles, and the following statement of its cost is given:‡

"Preliminary survey	$6 570.59
Engineering account	28 915.77
Expense account	7 322.75
Purchase at Lewis Wharf	23 215.84
Land damages and fencing	34 506.60
Salaries	12 062.98
Depot Wharves at East Boston	69 648.50
Depot at Salem	8 340.38
Cars and Engines	51 840.95
Grading, bridging, railway, East Boston ferry, etc.	589 478.94
	$831 903.30"

or about $64 000 per mile.

The Western Railroad reports§ 694 miles of surveys made between April and January, at a cost of $30 319.36, with $220 paid for land damages, which makes the average cost of surveys $43.69 per mile.

Later the same road reports‖ the cost of 54.57 miles of track, with 1.93 miles of turnouts and depot tracks, as $1 899 165.72; and gives the

*Massachusetts Railroad Reports for 1840.
†Massachusetts Railroad Reports for 1843.
‡Massachusetts Railroad Reports for 1838.
§Massachusetts Railroad Reports for 1836.
‖Massachusetts Railroad Reports for 1840.

cost of rolling stock, consisting of eight engines, seven 8-wheeled and six 4-wheeled first-class passenger cars, two second-class ditto, three baggage, seventy freight, ten dirt, and four hand, cars, in the sum of $117 804.18. The average cost per mile, including equipment, is thus seen to be $35 700. Mr. Wood.

The cost of 20.354 miles of the Boston and Maine was $431 592.15,* or an average of $21 204 per mile for construction. The cost per mile, including equipment, distributed over its trackage in Massachusetts and New Hampshire, appears to have been $22 666, which leaves a low figure for equipment.

The average cost of engines, cars, etc., per mile of New England roads, in 1841, was $3 022.†

The Boston and Providence, in 1854, paid $5 050 for a locomotive;‡ they paid $8 000 for the new one bought in 1859.§

The only salaries which the writer has been able to ascertain are those paid on the Fitchburg Railroad during construction.‖ The president, "who is also superintendent," received $1 500 per annum; the treasurer, $800; and the clerk and assistant to treasurer, $600. The engineers were paid $5, and the assistant engineers, $2 per day, paying their own expenses, except a small allowance for horse hire.

W. D. Taylor, M. Am. Soc. C. E., the author of this paper, died August 26th, 1911, before any discussion on his paper had been received.

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*Massachusetts Railroad Reports for 1843.
†Massachusetts Archives—Senate Document 17-1841, p. 83.
‡Massachusetts Railroad Reports for 1854.
§Massachusetts Railroad Reports for 1859.
‖Massachusetts Railroad Reports for 1843.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1207

ECONOMY IN RECTANGULAR PANELS, USING BEAMS OF CONSTANT CROSS-SECTION.

By J. S. Branne, Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. E. G. Walker and J. S. Branne.

It is claimed by the Engineer that he makes public improvements, or work, possible, by designing structures with the greatest economy consistent with safety, taking into account the first cost, maintenance, and probable life of each structure. In doing this he calls to his aid his entire store of knowledge and experience, in order that, in the general plan, the relatively useless and expensive features may be eliminated, and the truly essential and useful reasons for undertaking the work may be emphasized and kept constantly in mind.

Having arrived at a fairly well-planned working model, the detailed building of the parts proceeds along the lines of the general scheme, but on a smaller scale. At this stage the engineer must rely on his skill in grouping the parts, so as to give the best results for the money to be expended.

The object of this paper is to inquire into the most economical way of grouping the parts in a rectangular panel, such as is formed by two girders and two or more beams, the panel of a floor, or the floor-beams and stringers of a bridge.

Although engineering literature abounds with meritorious books on statics and applied mechanics, and with special works relating to structures, and although engineers are more or less familiar with the latest conceptions of strength as well as economy in bodies taken alone, whether they are beams, girders, columns, trusses, or arches, and whether they are simple or not so easily determinate, yet the writer has always been impressed by the fact that economy does

not consist simply in using the material in the several parts to their individual best advantage, but that certain relations of parts must be observed in order to obtain the most economical whole. It is a question of give and take, or of the economical balancing of parts. This presents a wide field for investigation, both theoretical and empirical; and, though this paper treats of only a modest class of parts which are quite readily analyzed, yet it is important in an economical sense.

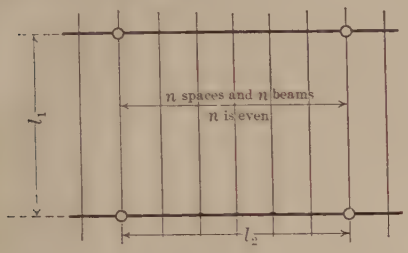


FIG. 1.

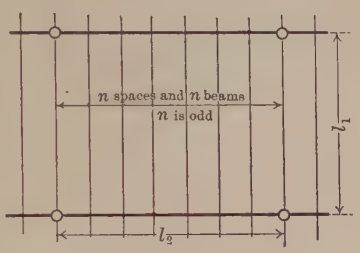


FIG. 2.

Select a rectangular panel from an indefinite number of contiguous panels of the same dimensions: Let its area be $K = l_1 l_2$, and let the center bending moments on the simply supported beams be M_1 and on the girders M_2. Then, for evenly-distributed loads (Figs. 1 and 2),

$$M_1 = w \frac{l_1^2 l_2}{8}, \text{ and } M_2 = w \frac{l_2^2 l_1}{8} \times \frac{n^2 - 1}{n^2}.$$

Here, w is in pounds per square foot, uniformly distributed, and n is the number of spaces in the panel, equal to the number of beams. When n is even, $\frac{n^2 - 1}{n^2} = 1$. If the price per foot-pound of M_1 is C_1 and of M_2 is C_2, where C_1 and C_2 have any values expressed in the same units of money, then $M_1 C_1 + M_2 C_2$ must be a minimum.

By "price per foot-pound" is here meant the cost of the material in beams or girders divided by the respective center bending moments. From this it follows that:

$$l_1 K C_1 + \frac{K^2}{l_1} C_2 = f(l_1) \text{ must be a minimum} \ldots\ldots (1A)$$

By taking the minimum value out of this expression and putting its first derivative equal to zero:

$$l_1 C_1 = l_2 C_2,$$

$$\text{whence } M_1 C_1 = M_2 C_2 \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

This result might have been foreseen, as the law of the sum of two functions of l_1; but Equation (1*A*) is developed because it is useful in plotting cost curves. Therefore, the economy in a rectangular panel, where the center bending moment determines the section, and the cross-section is constant, means that the cost of the beams should equal the cost of the girders; or,

$$\underset{\text{(Cu. in. in beams} \times \text{cost, 1.)}}{n \times a_1 \times l_1 \times 12} = \underset{\text{(Cu. in. in girder} \times \text{cost, } C\text{.)}}{C \times a_2 \times l_2 \times 12} \quad \ldots\ldots\ldots (2)$$

In order to apply this equation of condition, (2), it may be well to consider carefully what it implies:

First.—It gives the condition of greatest economy for two dependent series of bodies, subject to bending, as explained. The first series—the beams—has an aggregate cross-section per panel of $n \times a_1$ square inches; the second—the girder—has a cross-section of a_2 square inches. Therefore, when beams and girders of certain widths and depths, or weights and depths, are used—all beams being alike, and all girders alike—for the greatest economy, the respective quantities should be equal.

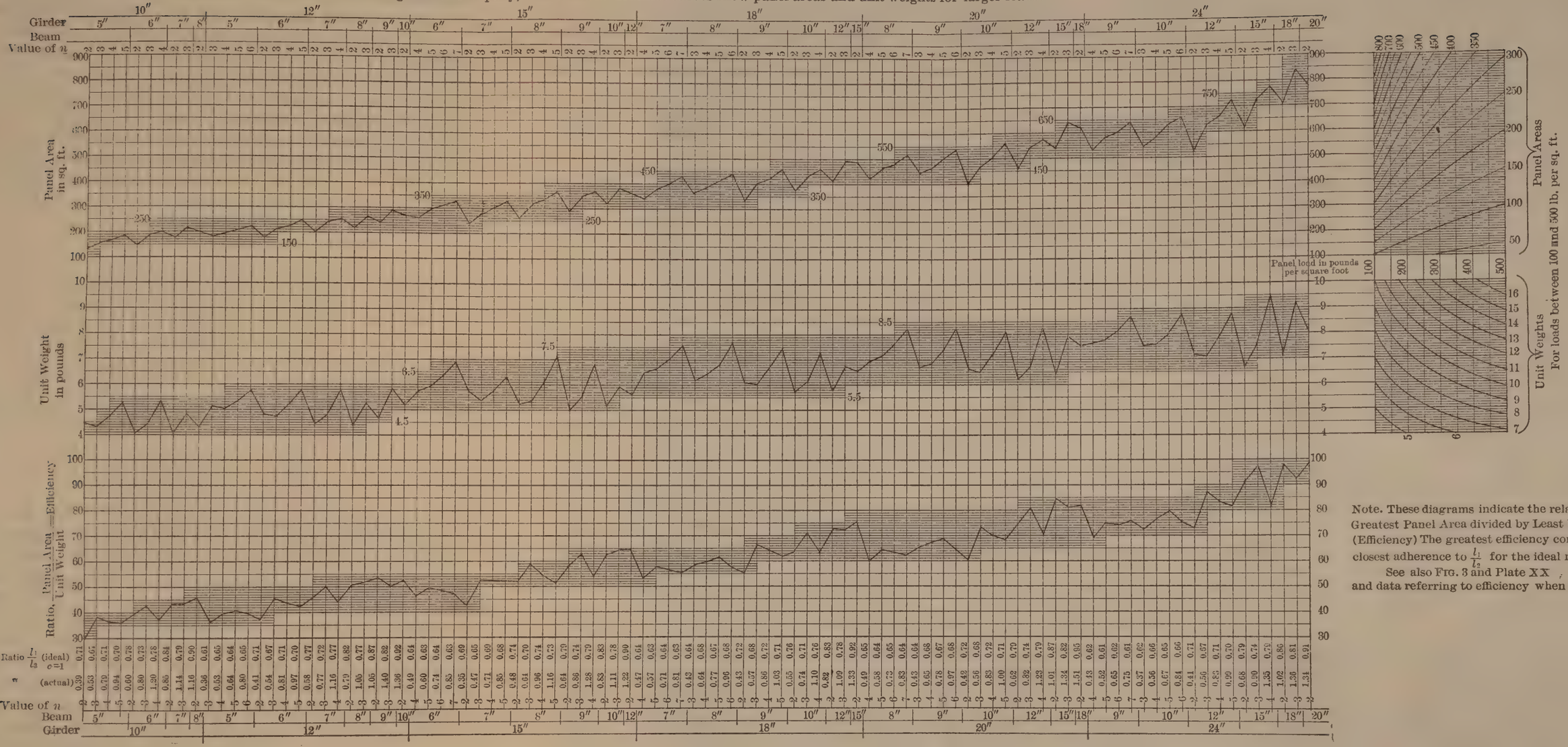
Second.—The uniformly distributed load, w, does not enter into the equation, hence the relation is independent of w. Any rectangular area, K, therefore, for any intensity of floor loading, has a certain economical ratio between its dimensions, l_1 and l_2, depending on the properties of the selected beams and girders; and, if this ratio is adhered to, K is the greatest area that can be had without increasing the quantity of material used per square foot of floor.

The ratio, $\frac{\text{Panel area}}{\text{Unit weight}}$, may be called the "efficiency," and clearly indicates economy.

In the practical application of the principle contained in Equation (2), the panel area, K, may be given, and such depths of beams and girders as may be determined by other considerations, as, for example, head-room; or l_1 and l_2 may be given, whence the ratio of depths of beams and girders can be found. Naturally, there are many other ways in which the problem may come up. Again, the ideal relation expressed by Equation (2) may be realized only in beams and girders made of plastic materials, as concrete; or of metals cast to proper size. If the beams and girders are of wood, the difficulty of reaching the ideal condition of Equation (2) is not great, as the widths can be varied,

DIAGRAM SHOWING PANEL AREAS, UNIT WEIGHT, AND EFFICIENCY, FOR VARIOUS COMBINATIONS OF BEAMS.

Uniformly distributed load, w=100 lb. per sq. ft. All beams minimum-weight sections, as per "Pocket Companion" of Carnegie Steel Company, 1903 edition. Transfer curves show panel areas and unit weights for larger loads.


Girder
Beam
Value of n
Panel Area in sq. ft.
Unit Weight in pounds
Ratio, Panel Area / Unit Weight = Efficiency
Ratio $\frac{l_1}{l_2}$ (ideal) c=1
n (actual)
Panel load in pounds per square foot
Panel Areas
Unit Weights
For loads between 100 and 500 lb. per sq. ft.

Note. These diagrams indicate the relation: Greatest Panel Area divided by Least Unit Weight. (Efficiency) The greatest efficiency conforms with closest adherence to $\frac{l_1}{l_2}$ for the ideal relation.

See also FIG. 3 and Plate XX, and data referring to efficiency when n=3.

As to the direct question of size of panel, the quantity, K, varies. The unit cubic contents of the panel may be put equal to

$$108\, w \left(\frac{l_1^2}{f_1\, h_1} + \frac{l_2^2}{f_2\, h_2} \times C\right).$$

This quantity increases and decreases with K, if the economic relation of l_1 and l_2 is constant. Of course, if there is a deviation from this, even a smaller panel may have greater unit cubic contents.

Regarding the value of n: naturally, in any panel, if the material apportioned to the beams can be divided by three, or $n = 3$, this makes the best possible arrangement for beams of constant cross-section, because, in that case, the entire middle third of the girder is used to its full capacity in bending.

In the ideal panel, when $n = 3$, there is a saving of about 5% over any arrangement in which n is even. When $n = 5$, the advantage of the odd number is almost negligible.

If rolled-steel beams are used (and proceeding in the same manner as with beams of rectangular cross-section), it will be found that:

$$l_2 = \sqrt[4]{K^2 \frac{r_2}{C\, r_1} \times \frac{n^2}{n^2 - 1}}$$

$$\text{and } l_1 = \sqrt[4]{K^2 \frac{C\, r_1}{r_2} \times \frac{n^2 - 1}{n^2}}$$

$$\text{whence } \frac{l_1}{l_2} = \sqrt{\frac{C\, r_1}{r_2} \times \frac{n^2 - 1}{n^2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (4)$$

Here, $\frac{n^2 - 1}{n^2}$, or its reciprocal, disappears when n is even. The ratio, $\frac{r}{h}$, has been assumed to be constant $\left(\frac{4}{10}\right)$. This is nearly right for sections of minimum weight. If certain beams and girders having radii of gyration, r_1 and r_2, are selected, the difficulty is that the girder seldom will match an integral number of beams. The inquiry may then best be directed toward finding which value of n brings the combination nearest to the ideal relation.

Using the general formulas for bending, and incorporating the relation in Equations (2) or (4):

$$N = \frac{S_2}{S_1} \sqrt{\frac{C\, r_1}{r_2} \times \frac{n^2}{n^2 - 1}} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (5)$$

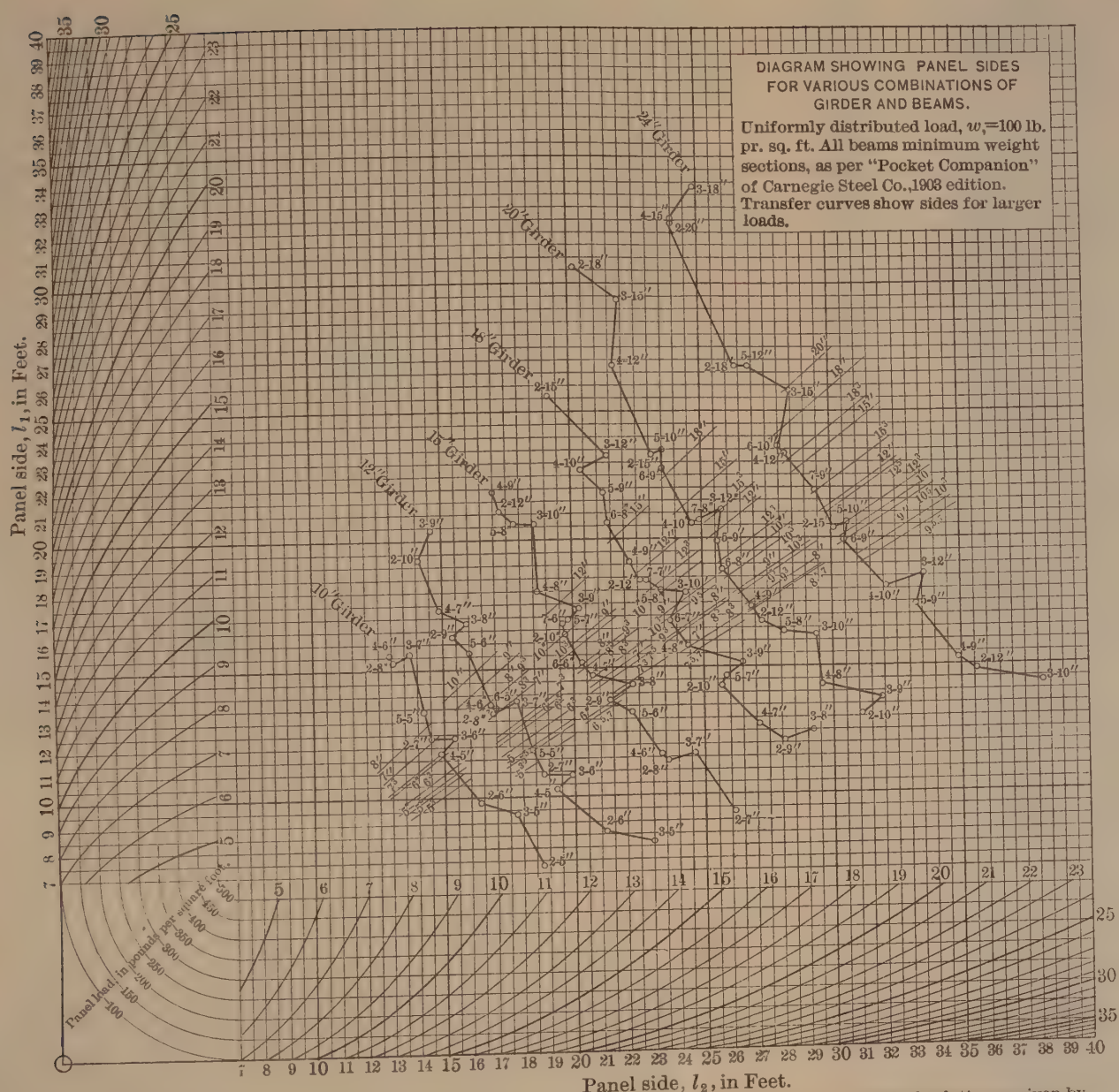


DIAGRAM SHOWING PANEL SIDES FOR VARIOUS COMBINATIONS OF GIRDER AND BEAMS.
Uniformly distributed load, w_1=100 lb. pr. sq. ft. All beams minimum weight sections, as per "Pocket Companion" of Carnegie Steel Co.,1903 edition. Transfer curves show sides for larger loads.
24" Girder
20" Girder
18" Girder
15" Girder
12" Girder
10" Girder
Panel side, l_1, in Feet.
Panel side, l_2, in Feet.
Panel load, in pounds per square foot.

Note. The object of this diagram is to show the divergence of several beam combinations from the ideal relation as given by $\frac{l_1}{l_2} = \sqrt{cr_1 \div r_2}$ when n is even, or $\frac{l_1}{l_2} = \sqrt{cr_1 \div r_2}\sqrt{(n^2-1) \div n^2}$ when n is odd. The ideal relations are drawn as fine lines.

Here, N represents the theoretical number of beams required in the ideal panel, and is generally fractional. S_2 and S_1 are the section moduli of girders and beams, respectively; and $\frac{n^2}{n^2-1}$ disappears when n is even. Therefore, the integral number nearest to N will be the best number of beams to use, bearing in mind that one should investigate the relations of one girder and several beams, all of definite properties.

If N is close to the numbers, 2 or 4, it is nevertheless quite likely that a three-space panel will give the greater efficiency $\left(\frac{\text{Panel area}}{\text{Unit weight}}\right)$. Similarly, if N is nearer to 4 or 6 than to 5, a five-space panel may show the greater efficiency, for the reasons stated previously. Selecting, then, this n which is close to N, the general formula for bending gives:

$$\text{(when } n \text{ is even)} \quad \frac{l_1}{l_2} = n\,\frac{S_1}{S_2}$$

$$\text{(when } n \text{ is odd)} \quad \frac{l_1}{l_2} = \frac{n^2-1}{n} \times \frac{S_1}{S_2}.$$

These are the actual relations, or as near as one can get to the ideal ones.

As to the value of C, this will probably be between 0.94 and 1.00, considering cost of fabricated material only; but if cost of erection is taken into account, the cost of the beams will be higher, owing to the placing of more pieces, and C may be as low as 0.80.

For intensities of floor loading other than w,

$$\frac{l_2}{w_1} = \frac{l_2}{w}\sqrt[3]{\frac{w}{w_1}}, \qquad \frac{l_1}{w_1} \quad \frac{l_1}{w}\sqrt[3]{\frac{w}{w_1}}$$

$$\frac{l_1\ l_2}{w_1\ w_1} = K = \frac{l_1\ l_2}{w_1\ w}\sqrt[3]{\frac{w^2}{w_1^2}}$$

$$\text{Unit weight}_{w_1} = \text{Unit weight}_{w}\sqrt[3]{\frac{w_1}{w}}.$$

When the shape of the panel is practically fixed by the prevailing conditions, the ideal relations may serve to indicate the most economical flooring, long or short span, tile or concrete. When the shape of the panel approaches a square, it may indicate long span construction.

Plates XIX and XX, and Fig. 3, show graphically the relations existing in rectangular panels, and these relations are illustrated by a

few combinations of rolled girders and beams. w has been assumed to be 100 lb. per sq. ft.

Plate XIX shows areas, weights, and ratios. The curves are self-explanatory, the upper two being drawn to illustrate the lower one, or the efficiency curve, which is the criterion for economy in a panel. The general efficiency of the three-space panel is quite evident, as explained previously.

Fig. 3 gives some cost curves, and illustrates the increment in total panel cost when the ideal relation, $\frac{l_1}{l_2}$, is deviated from in either direction. The "Total Panel Cost" is here plotted for various combinations of beams with a 15-in. girder, using Equation (1A). The following assumptions are made for the purpose of graphical illustration:

> Unit cost of girder = Unit cost of beam (or, $C = 1$);
> This unit cost has been put at 2 cents per lb.

Thus a 10-in. beam at 25 lb. per ft., in 1 ft. of its length, takes care of $(24.4 \times 1\,333) = 32\,500$ ft-lb. Here 24.4 is the section modulus. The cost of the beam per foot is $25 \times 2 = 50$ cents; or, the cost of the entire beam is $(50 \times l_1)$ cents. Therefore $(50 \times l_1)$ divided by 32 500 is the "Cost per foot-pound," as understood by C_1 in Equation (1A).

Similarly, for the 15-in. girder: $(84 \times l_2)$ divided by 78 700 is C_2. Introducing this in Equation (1A), we get

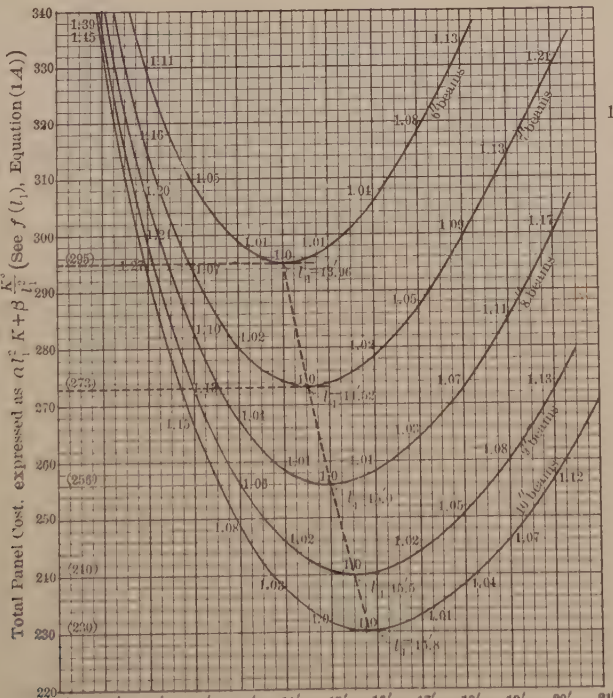
$$l_1 K \frac{50\, l_1}{32\,500} + \frac{K^2}{l_1} \times \frac{84\, l_2}{78\,700} = f(l_1),$$

which reduces to

$$\frac{1.54\, l_1^2\, K}{1\,000} + \frac{1.065\, K^3}{1\,000\, l_1^2} = f(l_1).$$

The cost for the 9, 8, 7, and 6-in. beams is found in a similar manner.

The "Total Panel Cost" at the left of Fig. 3 is in cents, and must be multiplied by $\frac{w}{8}$ for practical use. Note that these curves are plotted for n even. On Plate XIX it will be noticed that the only 10-in. relation plotted for n even is $n = 2$; also that the relation, $\frac{l_1}{l_2}$, is the ideal one; hence the cost at the bottom of the curve on Fig. 3 gives 230 cents with $K = 300$.



15″ Girder with 10″, 9″, 8″, 7″, 6″ beams. n even.
K assumed at 300 sq. ft.

	Values of α.	Values of β.
10″ I	1.54 $\frac{1}{1000}$	1.065 $\frac{1}{1000}$
9″	1.67 "	"
8″	1.90 "	"
7″	2.16 "	"
6″	2.52 "	"

Unit price of Girder assumed the same as Unit price of Beams. ($c=1$).
Min. Wt. Beams.

This diagram is a representation of comparative costs. Note the percentage of increment, showing how the relation, $n=3$, can outweigh a deviation from the ideal relation (bottom of curve).
The curves are plotted for n even.

FIG. 3.

On Plate XIX, the curve for "Panel Areas," using the transmission curve on the right, indicates a unit load of 112.5 lb. From the "Unit Weight" curve it is found that this corresponds to a weight of $5\frac{1}{3}$ lb., or 1 600 lb. for a panel area of 300 sq. ft., which, at 2 cents per lb., makes \$32. Now, from Fig. 3, 230 cents $\times \frac{112\frac{1}{2}}{8} =$ \$32.33. This is fairly close, for the small scale on which the diagram is plotted, and is one more example of the economy of graphical work. The same conformity will be found by actually computing the quantities from Plate XX.

Considering now the relation when $n = 3$, it is noted that the ideal panel relation is violated, giving $\frac{l_1}{l_2} = 1.11$, instead of 0.78; yet the efficiency is greater. Fig. 3 shows a rise in cost from 1 to 1.045, or $4\frac{1}{2}\%$, which is slightly outweighed by the intrinsic cheapness of the three-space panel, which gives a saving of one-ninth in the girder, or about 11%, or $5\frac{1}{2}\%$ on the whole panel weight, if weight of beams = weight of girders.

In this case the weight of the girder is 43% of the panel weight, or the saving on the entire panel is 4.77%, thus showing a slight advantage over $n = 2$.

Clearly, the three-space panel is the cheapest of all; the next is $n = 5$ and then $n = 7$, which latter, as to efficiency, approaches $n = \infty$, or uniform loading.

Plate XX shows several combinations, and indicates their greater or lesser economy as they approach or recede from the ideal relations, indicated by the fine lines drawn from the origin. The transfer curves are drawn from the relations previously stated.

It may be that where the combinations fall below the ideal ratio, $\sqrt{\frac{r_1}{r_2}}$, better results may be had by using weights greater than the minimum, for the beams; and, if above the ratio, by using girders of greater weight. These greater weights are generally (but not always) produced by spreading the rolls, thus resulting in smaller inertias, relatively, than the minimum weights, whence they are not quite as economical as the minimum weights, speaking now of bending only, and not considering their valuable properties in being better able to carry shear for heavy loads, short spans, or special considerations. By plotting a few, this could be easily determined.

DISCUSSION

Mr. Walker.

E. G. Walker, Jun. Am. Soc. C. E. (by letter).—The writer welcomes Mr. Branne's paper because, altogether apart from the detailed discussion of the case of rectangular panels, it serves to emphasize the fact that in all structures, of whatever type or magnitude, there is a relative proportion among the constituent parts which gives the most economical result. In planning new work, the designer's object should be to attain, to as great an extent as possible, this maximum of overall economy, even though, in some instances, this might require more expensive work in one particular item than if consideration had been limited to that item alone. The author recognizes this in his opening paragraphs, and, though the writer is in cordial agreement with the statement that this general subject of economy of design "presents a wide field for investigation, both theoretical and empirical," he is also of the opinion that, in general, the theoretical investigation of economy can only be of very limited use, because the conditions under which work is carried out differ so widely that it is hardly possible to fix on any set of general premises upon which to base an argument that can be applied equally to the variations which arise in practice. As an instance, a very ingenious theory was brought forward some years ago in connection with reinforced concrete design. The promoters of this idea started with the fallacy that the most economical design in any given case could be obtained by substituting in a general mathematical formula based on the so-called relative prices of concrete and steel. On this basis, they were able to devise mathematical formulas which would give what they considered to be the most economical percentage of reinforcement in any case and, at the same time, not exceed the prescribed limits of maximum stress. The results looked very promising at first glance, but, as a practical proposition, the method was untenable, and, as far as the writer is aware, was never taken up. No doubt it is possible to estimate with a fair degree of accuracy the cost of the reinforcement per ton delivered on the site, and the price per cubic yard for concrete at the mixer; but these figures are only remotely connected with the final relative costs of the two materials in the finished structure. It will be admitted, in general, that the cost of handling (including shuttering, etc.) the concrete in the vertical front slab of a retaining wall, or in a thin partition, is greater than that of putting the same concrete into ordinary beams and floor slabs. It is simply because of variations in the cost of labor, temporary work, supervision, etc., in the different components of a structure, that it is impossible in the general case to reduce economy in design to a rigid mathematical relationship. A lack of appreciation of the fact that economy of construction is a matter of practical experience, far more

Mr. Walker.

than of theoretical deduction, caused the failure of this cost theory of reinforced concrete.

This is only one instance, and many other examples can be easily found. The question of the most economical proportions between depth and span in an ordinary truss has been discussed by a number of authors, but the proportions in every-day use are entirely the result of the experience of bridge builders extending over many years; the writer believes that no satisfactory formula has been brought forward to settle these proportions. The attempts that have been made have failed, because economy in the construction of a bridge superstructure is something more than making the sum of the maximum areas of the numbers a minimum. It involves considerations of processes of fabrication and erection, appearance, trade practices, and even, in some cases, personal prejudices, which must outweigh the findings of any formula. Instances may be multiplied indefinitely.

Designing, in all branches of engineering practice, is a process of compromise, and it is only by an equitable balance of conflicting considerations that the truly economical result is attained. For this reason, the writer trusts that in any investigations carried out, as suggested in the paper, adequate regard will be given to the empirical considerations which, in all but the simplest cases, must predominate in actual practice over the purely theoretical.

The author has restricted his studies to the case of rectangular panels, uniformly loaded and supported by rolled steel joists, as he considers this to be one of the simplest cases, as most probably it is. For application he takes two cases: (1) the panels of a floor, and (2) the floor-beams and stringers of a bridge. Taking the second application first: It is apparent that there is no appreciable latitude in settling the dimensions of the floor panels of a bridge. The width of the structure is determined from *a priori* considerations, and the spacing of the floor-beams must be fixed mainly with reference to the economy of construction of the truss, taken in conjunction with the allowable size and weight of the floor-beams. There only remains to determine the spacing of the stringers. In bridges carrying railroads or street railways, this is practically settled in advance, though, in a highway bridge, there is a little latitude for the exercise of judgment. Even so, surely the weight and thickness of the flooring, which depend on the spacing of the stringers, have an influence on the cost. For example, the substitution of $\frac{1}{2}$-in. troughing instead of $\frac{3}{8}$-in., though perhaps having little influence on the total weight of the flooring to be supported in a given panel, will have a much greater influence on the cost. Yet, in Mr. Branne's investigation no account whatever is taken of the flooring, though this must have an effect on the cost which should not be neglected.

Mr. Walker.

In the case of the floor system of a steel-framed building (which the author appears to have had in mind) there is yet another disturbing factor. The weight and cost of a column of given length are by no means proportional to the load it has to carry. From general considerations, it would appear, in fact, that the cost per ton supported would decrease continuously with the load until a stage was reached at which the cost of handling and erecting assumed such greater relative importance than the saving of cost attained by putting a large load on one column as would cause a limit of economy to be reached, as far as the column considered by itself was concerned. Hence the influence of the division of a floor is carried into the column system supporting the floor, and the proposition becomes at once far more cumbrous to handle.

The introduction of the consideration of the columns is avoided in the paper by assuming the panel area, K, which the author states is to be "determined as some general size which appears to suit the case, as to magnitude." By so doing, the most important item in the economical lay-out of the structure is empirically settled, and the author's algebraic analysis and diagrams are relegated to a secondary position. Hence arises the question of the practical application of the paper. In general, the designer, when laying out a new building, is confronted with one of two conditions. Either he has unrestricted scope to place his columns where he likes, and thus divide his floors up into panels of whatever size he chooses; or, consideration of the requirements of the users of the building restricts him to certain positions for his columns, with a consequent unavoidable arrangement of the floor panels.

In the first case, the question of economy is complicated by having to consider the cost of the columns. Under the other condition, while it settles, usually within fairly close limits, the area, K, it also has the effect of leaving very little room for varying the ratio, $\frac{l_1}{l_2}$. Hence the opportunity of approaching the author's ideal ratio is but small.

Without in any way criticizing the author's mathematical analysis —a matter which is not now presumed—the foregoing considerations seem to show that the investigations given in the paper are only of limited application in practical work.

Mr. Branne.

J. S. Branne, Assoc. M. Am. Soc. C. E. (by letter).—The writer agrees with Mr. Walker that the application of the arrangement of panel sides to suit the depths of joists and their mechanical properties, and also involving the spacing of the columns, has a restricted application. The length of the columns enters into the problem, also the kind of flooring; but it is hardly possible to get a mathematical solution of all these matters. The paper, therefore, was restricted to a

Mr. Branne.

small area where it was possible to lay down a rational solution, many things being taken as settled *a priori*.

In the design of a tall building, the object is generally to get as much rental space as possible; hence as many floors as possible, and this reacts on the depths of the beams and girders not a little. The cost of the enclosing walls and of all vertical piping also enters the problem. It is to be hoped that some one will bring out a more comprehensive review than the writer has attempted, and it is conceded that such will of necessity involve much empirical work. The writer, in the latter part of the paper mentions the use of joists of equal size or equal properties as to their moment of inertia and area combined, and this would point to long-span construction, as in that case no beams could be required, theoretically, except at the columns, or the cost of the beams would equal that of the girders; or, if timber joists were used, several of the same depth could be utilized, the aggregate width of which would equal that of the girder.

The writer does not agree with Mr. Walker in his statement that the disposition of the columns has been relegated to a secondary place. If the designer knows, within narrow limits, the depths of the joists to which he is restricted, based on the head-room, etc., he can, from these known quantities, decide the panel lengths, taking a certain size as a basis for area, for example, 15 by 15 ft. = 225 sq. ft., or 18 by 18 ft. = 324 sq. ft., and, from the properties of the joists to be used, he can decide whether the area of 225 sq. ft. should be made up with sides 15 by 15 ft., or 13 by 17.3 ft., etc., aiming at all times to make the cost of the beams equal the cost of the girders. On the other hand, if the spacing of the piers or columns is already fixed, the sides of the panel will determine the properties of the beams and girders, and full economy can be had where the widths can be made to aggregate a certain amount, as in timber joists, to suit the equality, as stated.

The writer hopes that a more extended paper may be written, taking in a commercial building, where consideration may be given to the kind of flooring, the head-room (involving in itself the heating), the cost of the enclosing walls and of the partitions, the lengths of columns and piping, the cost of erecting the skeleton of the building, and other things allied thereto, as this would be an important contribution to the economy of building.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1208

ECONOMIC CANAL LOCATION IN UNIFORM COUNTRIES.*

By Lyman E. Bishop, Jun. Am. Soc. C. E.†

With Discussion by Messrs. H. J. Doolittle, C. E. Shipman, Sam G. Porter, C. C. Williams, and Lyman E. Bishop.

General Remarks.

As the writer has spent practically all his time, for the past three years, in the field location of canals and laterals, the method of economic location which has been developed may be of passing interest to others engaged in work of this class.

Yardage diagrams were made of standard canal sections with capacities of from 1 200 to 25 sec-ft. and a velocity of 2.5 ft. per sec. For capacities ranging from 1 200 to 250 sec-ft., the diagrams were of sections having intervals of 50 sec-ft.; for capacities of less than 250 sec-ft., a diagram was made for each interval of 25 sec-ft. Figs. 1 and 2 are typical sections, and Table 1 gives the dimensions of standard laterals having velocities of from 1.0 to 3.0 ft. per sec. and capacities of from 1 to 90 sec-ft.

In the yardage diagrams, Figs. 1 and 2, the center cut on a given transverse ground slope is used as the argument for the excavation and embankment for a required canal section, the elements of which section are predetermined. The quantities are given in cubic yards per station of 100 ft. The embankment quantities are 10% in excess

* Presented at the meeting of September 20th, 1911.
† Now Assoc. M. Am. Soc. C. E.

of the actual embankment, this being an arbitrary allowance for shrinkage and settlement. The center cuts vary from 6 ft. below the bottom of the canal to 16 ft. above grade, and the transverse ground slopes from the level to 35%, and is platted at 5% intervals.

Two embankment curves are platted for each different transverse slope; one gives the total yardage in the upper and lower banks combined; the other gives the yardage in the lower bank alone. These two curves join or coincide at the center-line cut, where the inside crown of the upper bank intersects the ground surface.

Whenever the full water level of the canal intersects the ground surface below the outside toe of the upper bank, the upper bank is discontinued. The note on each diagram (Figs. 1 and 2), "Do not stake out upper bank where water line intersects ground line within .... feet of center line," indicates where this condition prevails. This point is also indicated on the diagrams by a heavy horizontal line connecting the one-bank curve with the two-bank curve of the same transverse slope, and is supplemented by an explanatory note: "These breaks indicate center line cut at which upper bank is dropped."

Under ordinary conditions of regular location, greater center cuts require one bank only, while lighter center cuts require two banks. These requirements will always be governed by the full lines of the embankment curve. When, for any reason, the lower bank only is required, the dotted line below the horizontal line connecting the same transverse slope is used. When it is required to continue the upper bank beyond the point provided for under ordinary conditions, the dotted line above the horizontal connecting line is used.

Where the embankment curve for a given transverse slope intersects the excavation curve of the same transverse slope, the intersection center cut is called "economic" and is so indicated on the diagrams. At these intersections the yardages in excavation and embankment are balanced, allowance having been made for shrinkage and settlement of the embankment quantities.

At the right of each diagram a template of the canal shows the dimensions and the elements which govern the quantities given in the diagram. The elements, n, s, and v (Basin or Kutter formula), may be varied to meet the requirements of the engineer, and will determine the dimensions of the section required. The dimensions alone govern the quantities in any section diagram. It is only necessary, therefore,

to select a section which will meet the requirements of n, s, and v, and the quantities in excavation and embankment will be given in the diagram of the nearest corresponding section.

Estimates, prepared from these diagrams, using the center cuts and transverse ground slopes, from the final profile notes, meet every requirement of an economic location. This estimate will not vary more than 5% from the actual quantities, as determined from the cross-section notes. In making up preliminary estimates from preliminary lines, these diagrams are extremely useful.

Table 1 gives the standard sections having capacities ranging from 1.0 to 90 sec-ft. The different sections were computed on grades giving velocities of from 1.0 to 3.0 ft. per sec. The section having the required capacity and grade can be selected and varied as desired. In covering any particular tract of land, that grade will be selected which corresponds best with the general slope of the country. A lateral of a given capacity can be located on a grade which will give any velocity required between the limits of 1.0 and 3.0 ft. per sec.

Information Furnished to Location Parties.

The information furnished to location parties consists of diagrams showing the yardage in excavation and the yardage in embankment, plus 10% for the various slopes to be encountered. From these diagrams a table of economic center cuts for the given section and transverse slopes can be prepared.

Estimates are also furnished showing the comparative cost of earth (loose and solid rock classifications); siphons under different heads; the cost per linear foot of standard flumes; the cost per linear foot of trestles of different heights; and the cost of different classes of tunnels.

The locating engineer designs and selects the section best fitted for construction through the country in which he is locating. The grade of the section having the desired water area may be changed to give the required velocity, if the material through which the location is to be made warrants a velocity other than 2.5 ft. per sec.

Organization.

The organization of a locating party through uniform open countries consists of one locating engineer, one transitman, one levelman, one rodman, two chainmen, two stakemen, and a field draftsman.

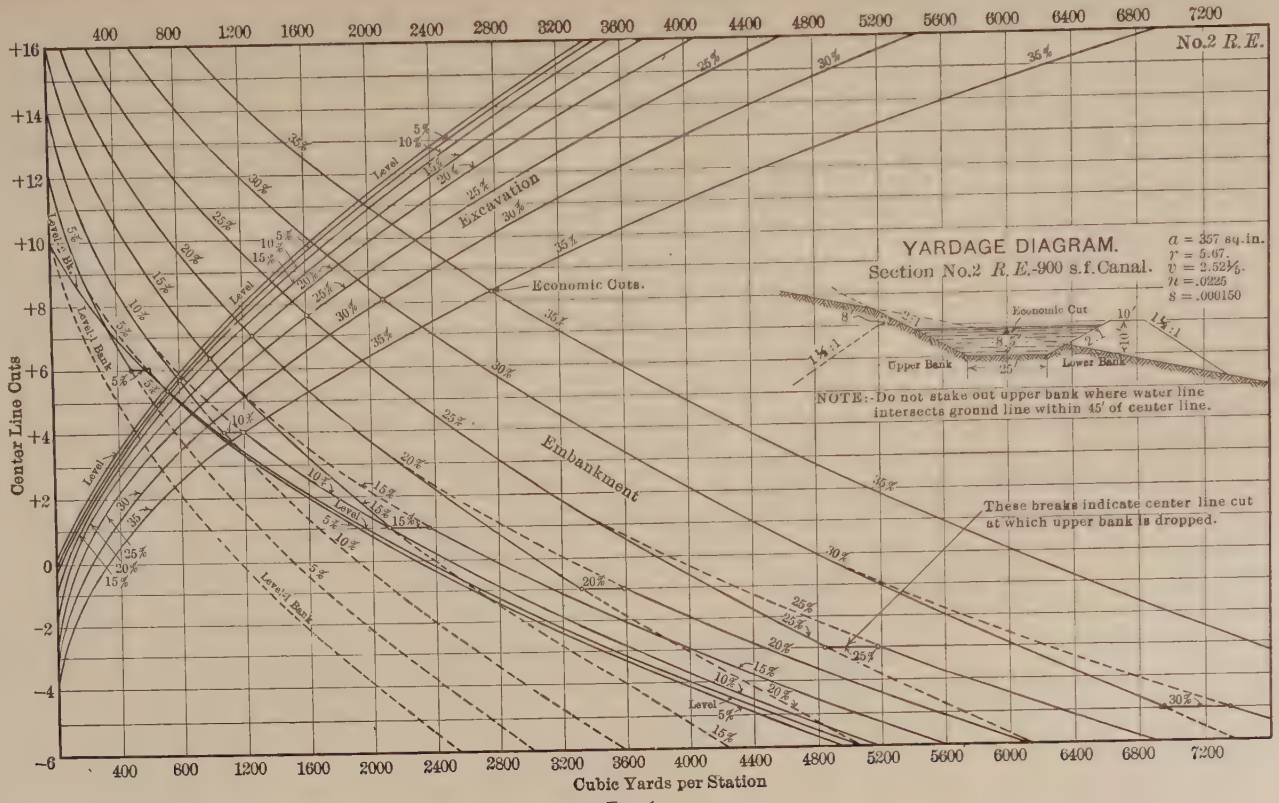

No.2 R.E.
YARDAGE DIAGRAM.
Section No.2 R.E.-900 s.f.Canal.
a = 357 sq.in.
r = 5.67.
v = 2.52⅓.
n = .0225
s = .000150
Economic Cuts.
Excavation
Embankment
Economic Cut
Upper Bank
Lower Bank
NOTE:-Do not stake out upper bank where water line intersects ground line within 45' of center line.
These breaks indicate center line cut at which upper bank is dropped.
Center Line Cuts
Cubic Yards per Station

FIG. 1.

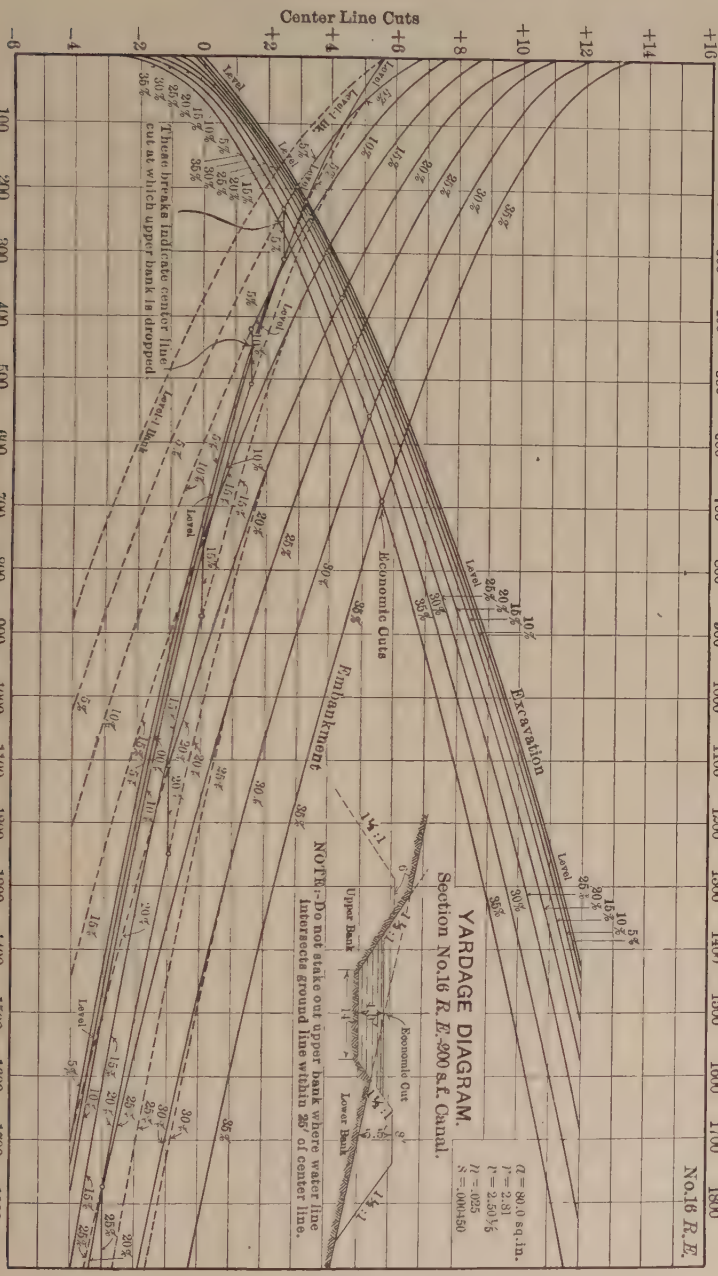

Center Line Cuts
YARDAGE DIAGRAM.
Section No.16 R. E. 900 s.f. Canal.
No.16 R.E.
Excavation
Embankment
Economic Cuts
Level
Upper Bank
Lower Bank
Economic Cut
NOTE:-Do not stake out upper bank where water line intersects ground line within 25' of center line.
These breaks indicate center line cut at which upper bank is dropped.
Cubic Yards per Station

FIG. 2.

TABLE 1.—(*Continued.*)

Section.	E. C.	0.75 : 1 000		1 : 1 000		1.5 : 1 000		2 : 1 000		2.5 : 1 000		3 : 1 000		4 : 1 000		5 : 1 000		6 : 1 000		7 : 1 000		8 : 1 000		9 : 1 000		Cubic yards, 100 ft. Sta.	B	F	T	W	P	A	R
		V	D	V	D	V	D	V	D	V	D	V	D	V	D	V	D	V	D	V	D	V	D	V	D							
36	1.6	1.70	28.7	1.95	32.9	2.40	40.5	2.75	46.4	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	67.6	9.0	1.5	4.0	1.50	14.4	16.87	1.17
37	1.6	1.70	30.0	2.00	35.2	2.45	43.2	2.80	49.3	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	70.5	9.5	1.5	4.0	1.50	14.9	17.62	1.18
38	1.6	1.70	31.2	2.00	37.7	2.45	45.0	2.80	51.4	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	73.5	10.0	1.5	4.0	1.50	15.4	18.37	1.19
39	2.0	1.70	22.7	2.00	26.7	2.45	32.7	2.85	38.1	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	59.3	5.0	1.5	4.0	1.75	11.8	13.35	1.18
40	1.9	1.75	26.6	2.00	30.4	2.50	38.0	2.90	44.1	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	62.3	6.0	1.5	4.0	1.75	12.3	15.20	1.23
41	1.9	1.80	30.2	2.05	34.4	2.50	42.0	2.95	49.6	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	69.3	7.0	1.5	4.0	1.75	13.3	16.80	1.26
42	1.8	1.85	34.6	2.10	39.3	2.60	48.7	3.00	56.1	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	71.3	8.0	1.5	4.0	1.75	14.3	18.70	1.31
43	1.8	1.90	38.6	2.15	43.7	2.65	54.0	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	78.0	9.0	1.5	4.0	1.75	15.3	20.35	1.33
44	1.7	1.90	42.0	2.20	48.6	2.70	59.7	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	79.0	10.0	1.5	4.0	1.75	16.3	22.10	1.35
45	1.7	1.90	45.3	2.20	52.4	2.70	64.3	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	85.3	11.0	1.5	4.0	1.75	17.3	23.80	1.37
46	1.6	1.95	50.0	2.25	57.6	2.75	70.5	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	85.3	12.0	1.5	4.0	1.75	18.3	25.60	1.40
47	2.2	1.85	29.6	2.15	34.4	2.60	41.6	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	67.6	5.0	1.5	4.0	2.00	12.2	16.00	1.31
48	2.1	1.90	34.2	2.20	39.6	2.70	48.6	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	71.2	6.0	1.5	4.0	2.00	13.2	18.00	1.36
49	2.0	1.95	39.0	2.25	45.0	2.75	55.0	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	74.1	7.0	1.5	4.0	2.00	14.2	20.00	1.41
50	2.0	2.00	44.0	2.30	50.6	2.85	62.7	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	81.5	8.0	1.5	4.0	2.00	15.2	22.00	1.45
51	1.9	2.05	49.2	2.35	56.4	2.90	69.6	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	83.4	9.0	1.5	4.0	2.00	16.2	24.00	1.48
52	1.9	2.05	53.3	2.40	62.5	2.90	75.4	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	90.4	10.0	1.5	4.0	2.00	17.2	26.00	1.51
53	1.8	2.10	58.8	2.40	67.3	2.95	82.6	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	91.3	11.0	1.5	4.0	2.00	18.2	28.00	1.54
54	1.8	2.10	63.0	2.45	73.5	3.00	90.0	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	98.0	12.0	1.5	4.0	2.00	19.2	30.00	1.56

In Kutter's Formula $n = 0.025$.

T = Width of Top of Bank, in feet.

D = Discharge, in cubic feet per second.

P = Wetted Perimeter, in feet.

E. C. = Economic Cut, Level Section.

W = Depth of Water, in feet.

B = Width of Bottom, in feet.

A = Area of Cross-Section, in square feet.

F = Distance, Top of Water to Top of Bank, in feet.

V = Velocity of Water, in feet per second.

All slopes 1½ : 1.

R = Hydraulic Radius.

TABLE 1.—Dimensions of Standard Laterals.
Velocities and Discharges for Different Slopes.

Section.	E. C.	0.75 : 1000		1 : 1 000		1.5 : 1000		2 : 1 000		2.5 : 1000		3 : 1000		4 : 1 000		5 : 1 000		6 : 1 000		7 : 1 000		8 : 1 000		9 : 1 000		Cubic yards. 100-ft. Sta.	B	F	T	W	P	A	R
		V	D	V	D	V	D	V	D	V	D	V	D	V	D	V	D	V	D	V	D	V	D	V	D							
1	1.1	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	2.0	2.3	2.2	2.5	12.8	1.5	1.0	3.0	0.5	3.3	1.125	0.34
2	1.1	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	2.0	2.8	2.2	3.0	2.3	3.2	14.9	2.0	1.0	3.0	0.5	3.8	1.375	0.36
3	1.2	....	....	1.00	2.3	....	....	1.4	3.3	....	....	1.7	4.0	2.0	4.7	2.3	5.4	2.5	5.9	2.7	6.3	2.8	6.6	3.0	7.0	16.9	2.0	1.0	3.0	0.75	4.7	2.34	0.50
4	1.2	....	....	1.00	2.7	....	....	1.5	4.1	....	....	1.8	4.9	2.1	5.7	2.4	6.5	2.6	7.1	2.8	7.6	3.0	8.2	....	....	19.1	2.5	1.0	3.0	0.75	5.2	2.72	0.52
5	1.2	....	....	1.10	3.4	....	....	1.5	4.5	....	....	1.9	5.9	2.2	6.8	2.4	7.4	2.7	8.4	2.8	8.7	3.0	9.3	....	....	21.3	3.0	1.0	3.0	0.75	5.7	3.09	0.54
6	1.1	....	....	1.10	3.8	....	....	1.6	5.5	....	....	1.9	6.5	2.3	7.9	2.5	8.6	2.8	9.7	3.0	10.3	....	....	....	....	21.0	3.5	1.0	3.0	0.75	6.2	3.45	0.56
7	1.1	....	....	1.10	4.2	....	....	1.6	6.1	....	....	1.9	7.3	2.3	8.8	2.5	9.6	2.8	10.8	3.0	11.5	....	....	....	....	23.0	4.0	1.0	3.0	0.75	6.7	3.84	0.57
8	1.1	....	....	1.20	5.0	....	....	1.7	7.2	....	....	2.0	8.5	2.3	9.7	2.6	11.0	2.8	11.8	....	....	....	....	....	....	25.1	4.5	1.0	3.0	0.75	7.2	4.22	0.59
9	1.0	....	....	1.20	5.5	....	....	1.7	7.8	....	....	2.0	9.2	2.4	11.0	2.6	11.9	2.9	13.3	....	....	....	....	....	....	24.1	5.0	1.0	3.0	0.75	7.7	4.59	0.60
10	1.3	1.00	4.5	1.30	5.8	1.60	7.3	1.8	8.1	2.1	9.5	2.3	10.4	2.7	12.2	3.0	13.5	....	....	....	....	....	....	....	....	23.8	3.0	1.0	3.0	1.0	6.6	4.50	0.68
11	1.3	1.15	5.7	1.30	6.5	1.60	8.1	1.9	9.5	2.1	10.5	2.3	11.5	2.7	13.5	3.0	15.0	....	....	....	....	....	....	....	....	26.3	3.5	1.0	3.0	1.0	7.1	5.00	0.70
12	1.3	1.15	6.3	1.30	7.1	1.60	9.0	1.9	10.4	2.1	11.5	2.3	12.9	2.7	15.1	....	....	....	....	....	....	....	....	....	....	28.6	4.0	1.0	3.0	1.0	7.6	5.50	0.72
13	1.2	1.20	7.2	1.40	8.4	1.70	10.2	1.9	11.4	2.2	13.2	2.4	14.4	2.8	16.8	....	....	....	....	....	....	....	....	....	....	28.0	4.5	1.0	3.0	1.0	8.1	6.00	0.74
14	1.2	1.20	7.8	1.40	9.1	1.70	11.0	2.0	13.0	2.2	14.3	2.4	15.6	2.8	18.2	....	....	....	....	....	....	....	....	....	....	30.2	5.0	1.0	3.0	1.0	8.6	6.50	0.76
15	1.2	1.20	8.4	1.40	9.8	1.75	12.2	2.0	14.0	2.25	15.7	2.5	17.5	2.9	20.3	....	....	....	....	....	....	....	....	....	....	32.4	5.5	1.0	3.0	1.0	9.1	7.00	0.77
16	1.1	1.25	9.3	1.40	10.5	1.80	13.5	2.0	15.0	2.30	17.2	2.5	18.8	2.9	21.8	....	....	....	....	....	....	....	....	....	....	31.2	6.0	1.0	3.0	1.0	9.6	7.50	0.78
17	1.3	1.30	9.5	1.50	11.0	1.90	13.9	2.2	16.1	2.45	18.0	2.7	19.8	....	....	....	....	....	....	....	....	....	....	....	....	28.6	4.0	1.0	3.0	1.25	8.5	7.34	0.86
18	1.3	1.35	10.7	1.60	12.7	1.95	15.5	2.25	17.9	2.50	19.9	2.75	21.9	....	....	....	....	....	....	....	....	....	....	....	....	31.1	4.5	1.0	3.0	1.25	9.0	7.97	0.88
19	1.3	1.40	12.0	1.60	13.7	2.00	17.1	2.30	19.8	2.55	21.9	2.80	24.0	....	....	....	....	....	....	....	....	....	....	....	....	33.5	5.0	1.0	3.0	1.25	9.5	8.59	0.90
20	1.3	1.40	12.9	1.60	14.7	2.00	18.4	2.40	22.1	2.55	22.5	2.90	26.8	....	....	....	....	....	....	....	....	....	....	....	....	35.9	5.5	1.0	3.0	1.25	10.0	9.22	0.92
21	1.3	1.40	13.7	1.60	15.7	2.00	19.6	2.40	23.6	2.60	25.5	2.90	28.5	....	....	....	....	....	....	....	....	....	....	....	....	38.3	6.0	1.0	3.0	1.25	10.5	9.84	0.94
22	1.2	1.45	15.2	1.70	17.8	2.05	21.4	2.50	25.6	2.65	27.8	3.00	31.4	....	....	....	....	....	....	....	....	....	....	....	....	36.9	6.5	1.0	3.0	1.25	11.0	10.47	0.95
23	1.2	1.45	16.1	1.70	18.8	2.05	22.7	2.50	27.2	2.65	29.4	3.00	33.3	....	....	....	....	....	....	....	....	....	....	....	....	39.1	7.0	1.0	3.0	1.25	11.5	11.09	0.96
24	1.2	1.45	17.0	1.70	19.9	2.10	24.6	2.50	29.3	2.65	31.0	3.00	35.2	....	....	....	....	....	....	....	....	....	....	....	....	41.3	7.5	1.0	3.0	1.25	12.0	11.72	0.98
25	1.2	1.50	18.5	1.80	22.2	2.10	25.9	2.50	30.9	2.70	33.3	3.00	37.0	....	....	....	....	....	....	....	....	....	....	....	....	43.6	8.0	1.0	3.0	1.25	12.5	12.34	0.99
26	1.5	1.50	14.0	1.80	16.9	2.15	20.1	2.50	23.4	2.75	25.7	3.00	28.1	....	....	....	....	....	....	....	....	....	....	....	....	34.7	4.0	1.0	3.0	1.50	9.4	9.37	1.00
27	1.5	1.50	15.2	1.80	18.2	2.20	22.2	2.50	25.8	2.80	28.3	....	....	....	....	....	....	....	....	....	....	....	....	....	....	37.5	4.5	1.0	3.0	1.50	9.9	10.12	1.02
28	1.5	1.55	16.8	1.80	19.6	2.25	24.4	2.55	27.7	2.80	30.4	....	....	....	....	....	....	....	....	....	....	....	....	....	....	40.3	5.0	1.0	3.0	1.50	10.4	10.87	1.05
29	1.8	1.55	18.0	1.85	21.5	2.25	26.1	2.60	30.2	2.90	33.7	....	....	....	....	....	....	....	....	....	....	....	....	....	....	54.7	5.5	1.5	4.0	1.50	10.9	11.62	1.07
30	1.7	1.60	19.8	1.85	22.9	2.25	27.8	2.60	32.2	2.95	36.5	....	....	....	....	....	....	....	....	....	....	....	....	....	....	53.8	6.0	1.5	4.0	1.50	11.4	12.37	1.08
31	1.7	1.60	21.0	1.90	24.9	2.30	30.2	2.65	34.8	3.00	39.4	....	....	....	....	....	....	....	....	....	....	....	....	....	....	56.9	6.5	1.5	4.0	1.50	11.9	13.12	1.10
32	1.7	1.65	22.9	1.90	26.4	2.30	31.9	2.65	36.8	3.00	41.6	....	....	....	....	....	....	....	....	....	....	....	....	....	....	60.1	7.0	1.5	4.0	1.50	12.4	13.87	1.12
33	1.7	1.65	24.1	1.90	27.8	2.35	34.4	2.70	39.5	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	63.3	7.5	1.5	4.0	1.50	12.9	14.62	1.13
34	1.7	1.65	25.4	1.95	30.0	2.40	36.9	2.70	41.5	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	66.4	8.0	1.5	4.0	1.50	13.4	15.37	1.15
35	1.6	1.70	27.4	1.95	31.4	2.40	38.7	2.75	44.4	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	64.6	8.5	1.5	4.0	1.50	13.9	16.12	1.16

Method of Location.

Level Party.—The level party, consisting of one levelman, one rodman, and one stakeman, precedes the location party. The rodman carries the stationing by pacing, following the grade contour as nearly as possible. At each station, or at closer intervals, depending on the irregularity of the country, the rodman gives two readings on the ground, 50 ft. apart, at right angles to the line of the proposed canal. These two readings give the transverse slope of the ground. The levelman then sets the rodman at a point, the elevation of which will give the desired economic center cut for that particular ground slope. The level party proceeds in this manner, setting the lath, flagged with white, not more than 100 ft. apart. These preliminary levels are carried very carefully, and, at this time, construction bench-marks are established, which are adhered to in running the final profile, in cross-sectioning, and in staking out structures.

At points where it is evident that a saving in distance and material is possible, by cutting through ridges or fills across gulches, the levelman runs a profile line on the cut-off, in addition to his contour line. From his section diagram comparative estimates of the two lines are made. In case the shorter line proves more economical, a row of red flags (red meaning off grade) is set on the economical location. Both lines are left in, and the estimate on each line is marked on its first flag.

The preliminary level line is run not more than $\frac{1}{2}$ or $\frac{3}{4}$ mile ahead of the location party, on account of the difference in stations between the rodman's pacing and the measurement by the location party. This difference might be enough to affect the economic cuts due to the allowance in grade made by the levelman.

It has been found that the level party, in addition to setting the preliminary lath ahead of the transit party, is also able to take the final profile of the located line as it is run in.

Transit Party.—The transit party, in charge of the locating engineer, proceeds as follows: Being ahead of the transit party, the locating engineer picks out a tangent which will fit the flags established by the levelman. It is his endeavor, not to follow exactly the contour as shown by the levelman, but to lay a line which balances the yardage in excavation and embankment within the limits of economic haul. Having established such a tangent, the chainmen proceed along it,

setting the stakes not more than 50 ft. apart. The locating engineer proceeds ahead to the point of intersection of the next curve necessary. As soon as the chainmen have set the stakes to within the distance estimated as the semi-tangent for the next curve, a P. C. is put in and the transitman is called up. The chainmen then measure the semi-tangent to the point of intersection, as established by the locating engineer. From this P. I. they chain the same distance along the other tangent of this curve, being lined in on the tangent by the locating engineer. This will give the approximate P. T. of the curve required. The transitman reads the angle from P. C. to P. T. and calculates the degree and length of curve to swing into the next tangent. The chainmen then proceed setting stakes on this curve and establish a hub with a tack at the P. T.

The field draftsman plots the alignment notes and section line ties, and makes the right-of-way map. He also prepares and finishes all profiles and estimates of quantities, including the quantity and direction of hauled material, keeping his work up as fast as the notes are turned in from the field. No estimate is placed on the profile until it is approved by the locating engineer. Any piece of location which does not balance up well in excavation and embankment, within the limits of economic haul, after the estimate on it has been prepared, is relocated.

In country where shale or rock comes close to the surface, a testing party is put on. In cases where the rock is so close to the surface that the economic cut would necessitate the excavation of some rock, the location is made so that there will be a minimum of rock excavation, it being cheaper to borrow the material for embankments than to make the rock excavation.

At points where it is necessary to have structures, such as drops, culverts, siphons, and highway or railway bridges, the locating engineer should choose the most economical location under the circumstances. He also draws detailed sketches in his notebook of the kind of structure best fitted for that location. In addition, topography is taken, showing accurately the lay of the surrounding country within the possible limits of changes in location.

Profile.—The level party takes the final profile, showing all the breaks in the ground, also the slope of the ground at right angles to the located line. The final profile and alignment notes are turned

over to the field draftsman, and the notes and estimates of quantities are worked up.

Curvature.—The maximum curvatures allowed for canals of different sizes in earth are as follows: For canals with a carrying capacity of less than 400 sec-ft. the minimum radius permissible is six times the bottom width; for canals carrying from 900 to 400 sec-ft. the minimum radius is ten times the bottom width; and for capacities from 900 to 1 500 sec-ft. the minimum radius is twelve times the bottom width.

Curves of moderately long radius are not detrimental in the operation of any canal, and they should be used as far as possible to lighten the expense of construction. In locating around sharp rocky points, where the radius of curvature used is less than the allowable, the water slopes of the canal must be paved or rip-rapped for an adequate distance, in order to preclude the possibility of erosion.

In open country, where there is no timber to contend with, location by the above method costs from $45 to $65 per mile. This covers engineering salaries and expenses.

When the location on any particular canal has been finished and the estimates and profiles have been worked up, the data are sufficient to let the contracts. Until the line is cross-sectioned and the final quantities are worked up, the contractors are paid for work completed from the location quantities. The difference between the two estimates is balanced when the final measurements are made.

DISCUSSION

H. J. Doolittle, Assoc. M. Am. Soc. C. E. (by letter).—After having spent some six years on railroad location work, and about two years on canal location with the United States Reclamation Service, the writer is very much interested in the methods described in this paper, and quite agrees with them in general. In some instances he has adopted slightly different methods, which are presented herewith. Mr. Doolittle.

Before going into the field, tables of semi-tangent lengths, tangent offsets, external secants, normal cuts, and excavation quantities, were prepared. The organization of the party was the same as described by Mr. Bishop, except that a back-flagman and only one stakeman were employed. The tables were distributed as follows:

To the chief of party, the tables of normal cuts and quantities;
To the transitman, the tangent offsets;
To the head chainman, the semi-tangent lengths, curve lengths, and externals.

As chief of party, the writer carried a level rod, clinometer, signal cloth, and a piece of profile paper for the purpose of carrying grade elevations.

By the use of the level rod and clinometer, the slope was obtained, and, from the table of normal cuts, the desired normal cut, which, added to the grade elevation, gave the elevation of the desired ground surface. The rod reading was then calculated, the elevation of the levelman's H. I. being known, the target was set, and then the levelman was signaled to set the writer on grade. This point was witnessed by a piece of signal cloth.

In the writer's judgment, these grade points should be set by the chief of party, who, while doing so, can best study the ground; and it will only be necessary for him to establish grade at controlling points. The change of grade per station is usually so slight that a variation of 100 ft. in distance is of no consequence, and hence, the distances ahead to grade points can be paced with sufficient accuracy for all practical purposes.

When the central angle will justify it, the writer prefers to run to the P. I., which is established by the chief of party, who, from the tangent ahead, halts the head chainman, as he advances along the back tangent, when he reaches a point which will best fit the tangent that is being established. The transitman then sets up at the P. I., measures the central angle, which, with but a slight change, can often be made either an even degree or an even 10′, thus affording easy use of the tables.

The transitman announces the central angle, and the degree of curvature is selected; this is determined usually either by the de-

Mr. Doolittle.

sired external secant or the proper semi-tangent length. The head chainman then calls out the semi-tangent length and the curve length from the table, and he and the transitman calculate the P.C. and the P. T., the stations of which are recorded on the witness stakes by the "stake artist." The P. C. is then set, and the first half of the curve is established by tangent offsets. Then the chainmen measure the semi-tangent distance to the P. T. and back in the remainder of the curve, also by the tangent offset method, and the distance between the last two stakes set in opposite directions is checked, and, if found correct, the chaining is resumed from the P. T.; thus, from one set-up, the curve is established in a rapid and practical manner.

When it is impracticable to establish the P. I. on account of the size of the central angle, and when the radius of the maximum allowable curve is less than 100 ft., the following method has been used successfully: Place the rear chainman as near the center of the curve as it is possible at first to locate him. With a little practice he will be able to judge pretty well when he is equidistant from the head chainman, who occupies the P. C., and the chief of party, who stands at the P. T. The head chainman then, holding the proper radius length, the other end of the chain being held by the rear chainman, moves along the curve as a trial, and, if he strikes above or below the desired tangent at the P. T., the rear chainman then takes a new position, so as to bring the head chainman on tangent to the line ahead, the rear chainman at the same time maintaining the proper distance from the preceding tangent. The head chainman then swings back to the corrected P. C. which can be established at an even plus, to facilitate calculating the deflections, which the writer always makes according to the rule that three-tenths of the degree of curvature equals the deflection in minutes per foot. The transitman then moves up to the P. C., and the curve is established by deflections, it being best to hub about half way on the curve.

The P. T. is determined by the following trial method: First, set the head chainman's flag at a point which seems to be the proper P. T., then measure over, from the last stake, the tangent offset distance. These two points give the direction of a tangent ahead. By this method the P. T. is shifted so as to give the desired direction for the tangent ahead, and the transitman again moves up to the P.T., and the work continues.

Mr. Shipman.

C. E. Shipman, Assoc. M. Am. Soc. C. E. (by letter).—This paper furnishes some useful information on a part of irrigation work to which all textbooks, as far as the writer has noticed, give but scant attention. They give lengthy discussions on dams, run-off, headgates, and diversion weirs, subjects which are no doubt of great interest to the Chief Engineer, but when the young engineer wants to

find out how to go into the field and actually locate a canal economically, he will find little to aid him in textbooks on irrigation engineering. Mr. Shipman.

This paper is the first the writer has seen on the details of canal location. It brings up a practical question which neither the author nor the textbooks mention, yet it is one which has frequently puzzled the writer, that is, how to handle overhaul. The writer has used practically the same method of location as that described by Mr. Bishop, and has found that in trying to fit a set of curves and tangents to a falling contour defined by flags, it is almost impossible to make the cuts and fills balance exactly. One station may have an excess cut of 30 yd. and the next an excess fill of the same amount. Will the contractor haul that excess cut of 30 yd. from one station to the next? He will not, but will waste in one station and borrow in the other. If the free haul is 300 ft., neither will he pick up 50 yd. of excess excavation in one station and haul it 290 ft. to fill up a hole in the bank of a station that has an excess fill. Neither will he pick up 100 yd. of excess cut in one station and haul it 100 ft. beyond the free-haul limit for 1 cent per yd., which is what he usually gets for overhaul. Yet he is expected to, and is paid for doing just those things. One cent per yard for overhaul may be proper on railroad work where a 100 000-yd. cut is to be hauled ½ mile or so and dumped into a short heavy fill, but it is not right for the small hauls usually encountered on canal construction. Neither is it right to expect the contractor to move small yardages for short distances, for he cannot do it economically. It has been argued that this is simply a method of paying the contractor, but it seems absurd to pay him for doing one thing when he actually does something else. On the other hand, if an attempt is made to locate the canal so that there will be no stations in which the fill exceeds the cut, a large quantity of material will have been wasted, and the cost of the earthwork will have been greatly increased. This is no doubt a minor difficulty of irrigation construction, but it is one about which the writer would like to have a little information.

In some cases, elaborate plane-table sheets have been prepared, and a paper location has been worked up on them to be run in later in the field. On one location party with which the writer was connected. this plan was tried. It worked like a charm until a mistake of a foot or so in the levels run by the plane-table party was found, when the paper location, representing an outlay of several hundred dollars, was discarded, and a method quite similar to that described by the author adopted. Plane-table sheets of the locations of important structures are very desirable, but, in ordinary rolling country, the experienced locating engineer can get along very well without them.

Mr. Shipman. The author states that monthly estimates can be made up from the cross slopes and the center line levels, to be corrected later in the final estimate. The writer has had some experience along that line. Such a scheme would work nicely were it not for the sub-contractor. He does a few stations, is paid by the contractor from data given in the monthly estimates, and leaves the country. On the final estimate, the sub-contractor's yardage is cut down a few hundred yards, and the Engineer is called on for some embarrassing explanations as to how it all happened. The writer believes it is better to hold up the monthly estimate a few days, rather than give out quantities which have not been checked.

Mr. Porter. Sam G. Porter, Assoc. M. Am. Soc. C. E. (by letter).—The writer wishes to emphasize the necessity of location with a view to economic maintenance as well as economic construction. It is an unfortunate fact that the majority of irrigation canals are located by engineers who have had no experience in the operation of an irrigation system; such being the case, the problems of operation and maintenance do not appeal to them. For instance, the writer's experience in location work preceded any experience in operation, and, as a consequence, he was not qualified to foresee some of the factors which now impress him as being among the most important.

In the past few years railroads have spent millions of dollars in improving their alignment in order to secure more economical operation. It is more difficult to figure in dollars and cents the advantage of a good location over a poor one, when applied to the operation of an irrigation canal, than when applied to that of a railroad. Nevertheless, the advantages are no less evident to the experienced irrigator.

Many faults are due to the old method of following the contour or flow gradient, no matter where it leads. Unfortunately, this method is still used by many who call themselves irrigation engineers. Their sole idea is to locate a line where the cuts will just balance the fills. There are hundreds of instances where this idea has been responsible for a series of short curves or waves which, without much additional cost, could have been largely or wholly avoided. Such a line was made to "fit the ground," but not to meet the requirements of economical operation. Curves are a source of constant trouble and expense. They cut on the outside and fill on the inside, both operations causing heavy maintenance expense, one requiring bank protection or rip-rap, and the other frequent cleaning out in order to preserve the full section of the canal.

At the present time, the deposit of silt which has formed in the Amity Canal is being dredged out. This operation is made necessary largely by the poor alignment of the canal, which, especially in the first 20 miles, has a great many sharp curves, on the inside of which

heavy deposits form. The alignment is being improved to a certain extent, but many changes which might have been made easily when the canal was first located, are now impracticable. The full value of such changes cannot be stated in dollars and cents where it is merely a matter of alignment, but, where the saving in distance and in grade are the principal items, many comparisons of value may be made. For instance, a recently proposed cut-off on the Amity Canal showed a saving of about 1 mile in distance and 1 ft. in grade, at a cost, in dredge work, of about $1 000 more than the cost of cleaning out the old line. In recommending this cut-off, the writer estimated the loss of water in the canal to average about one-half of 1% per mile. The Company runs approximately 100 000 acre-ft. of water per year. One-half of 1% of 100 000 acre-ft. is 500 acre-ft., and as the cut-off is near the head of the canal, and practically all the supply passes this point, 500 acre-ft. of water would be saved annually. This is in addition to the gain in grade (which, in this particular case, is very important), and to the improvement in alignment, as three sharp curves with weak banks are eliminated. On the strength of this statement, the cut-off was approved. Mr. Porter.

Where it can be done, it is much better to have the canal well in the ground, with embankment only on the lower side, instead of having a double embankment. Objection is made that the water backs out on the upper side and covers so much ground above the actual canal section that the loss by seepage and evaporation is excessive. This may be true in some instances; on the other hand, where an embankment is used on the upper side to confine the water, it is washed out by nearly every local rain, and, unless it is repeatedly repaired, the water spreads over the area which it would cover were the embankment not there. This will possibly call for the excavation of some excess material. It can be well used, however, in making an extra strong lower bank, with plenty of allowance for erosion, settlement, and general wear, and with a good road on top. It pays to have a good road on the ditch bank, so that the ditch rider will have no excuse for avoiding it, and the closer attention which the ditch will thus secure is worth a great deal. At present, when canal superintendents and ditch riders are using automobiles and motor cycles, certain stretches of the canal where the road on the bank is not good, are sure to be neglected.

In preliminary location work, the plane-table is not used as much at it deserves. It is a very useful instrument, the merits of which do not seem to be recognized outside of the Government service. Some engineers consider it a hobby of the United States Geological Survey and the Reclamation Service, and for that reason are prejudiced against it. Where accurate topography is required, however, it possesses advantages over any other method, and this is particularly

Mr. Porter.

true where it is desired to secure field information on which to base office projection and estimates to be worked up during the winter, when field work is impracticable. Good plane-table sheets give all the information necessary for making comparative estimates of alternative alignments. The transverse surface slopes are shown at all points, and Mr. Bishop's yardage diagrams can be used in connection with them to good advantage. The use of curve templets on a plane-table sheet will also prove much more economical, both in yardage and in time consumed, than the ordinary method of trying to fit the curve to the ground in the field. Furthermore, by showing, on the plane-table sheet, the ties between the projected and the preliminary locations, several parties can take up the line at various points, when field work is resumed in the spring, and a decided saving can be made in the time required to get the final line ready for contractors to begin operations along the entire distance.

The writer is at a loss to understand the reason for Mr. Bishop's mode of procedure in laying out curves. The P. I. being established by the locating engineer, he assumes the P. C. This establishes the semi-tangent distance. Then, when the chainmen measure this same distance from the P. I. along the other tangent, he states that this gives him the approximate P. T. of the curve. Why is it not the exact P. T.?

By calculating the curve from an assumed semi-tangent distance, as outlined by Mr. Bishop, one is sure to have an uneven or fractional degree of curvature, with its consequent inconveniences in calculations and instrument work. Would it not be simpler, and enable the engineer to fit the ground better, to call the transitman up to the P. I. instead of the P. C., measure the intersection angle, and choose the nearest full degree of curve to meet the requirements? The writer's point is that the P. C., or the semi-tangent distance, is not the most logical thing to assume, from which to derive the other functions of the curve.

The diagrams, tables, and suggestions which Mr. Bishop presents are very helpful to irrigation engineers, and will be gratefully received.

Mr. Williams.

C. C. Williams, Jun. Am. Soc. C. E. (by letter).—In presenting this paper to the Society, the author has performed a valuable service in that he has described certain field methods in canal location which are of assistance to the engineer. The method of finding the economic cut is much more elegant than any involving the shifting of a transparent outline of the canal section over the plotted cross-sections of the topography. These or similar curves have been used by many engineers in the West. Incidentally, it may be noted that the curves are not extended to cover transverse slopes greater than 35%, although

this might easily be done. Many canals are located on slopes greater than 35% and less than the natural slope of most earths, namely, 1½:1, 35% being about 3:1. Mr. Williams.

Owing to the extreme scarcity of water in most regions requiring irrigation, and the fact that the chief difficulty is in water shortage and not in land shortage, it would seem that the conservation of the water should be the chief factor in discussing the economic location of canals. If a canal is wasteful of water and fails to carry it to the most advantageous points, it will hardly be economically located, regardless of the cheapness of construction. In an analogous manner, a railroad, located so that it may be constructed at minimum expense, may fall short of its main purpose if its route fails to reach the centers of population. In other words, in canal location, as in most matters, the cheapest is not always the most economical. A ditch located by using the grade contour and economic cut may follow the topography so closely that it will lose by absorption and evaporation a large percentage of the water carried.

A few years ago, while making some observations on irrigation ditches in the northern part of Colorado, with a view to obtaining information concerning the flow of water in such channels, the writer found that in many cases they were injured by excessive curvature. The section at the curve had been washed into an irregular outline, there being a deep pool on one side and a shallow grass-grown shore on the other. Such pools greatly increase the total absorption and evaporation losses. Mr. Bishop's practice of establishing as long tangents as possible by balancing cuts and fills along the line, within the limit of economic haul, is strongly to be commended, for by so doing the curvature can be reduced to a minimum. A plotted profile and a paper location in rough country may frequently be the best means of accomplishing this result. The excess of fall may be taken up by drops, chutes, etc.

Another danger in following the grade contour too closely by using the economic cut—or, as one might say, locating by "rule of thumb"—is the failure to take advantage of controlling features of topography, such, for example, as making a deep, short cut or a tunnel through the upper end of a projecting ridge, thus saving a long detour around such an elevation. Assuming a loss due to evaporation and seepage of ½ sec-ft. per mile, which is not infrequent, and a duty of 100 acres per sec-ft., the serious disadvantage of any increase in length of line becomes apparent. In a similar manner, a trestle, or an inverted siphon across a drainage channel, may save a detour around the head of such a waterway. For the purpose of cheapening construction, canals have been built, unfortunately, where the lower embankment only was constructed in crossing shallow draws, thus allowing the water

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to back up the natural channel. The added loss due to such construction is frequently enormous, for the bottom is very pervious, and permits a rapid rate of seepage.

Table 1 uses the same value of n in Kutter's formula for ditches of 2.3 sec-ft. capacity as for canals carrying 90 sec-ft. The writer's observations, as well as those of several others, notably Fortier and Flynn, show that n is not a function of the character of the bed only, but depends to some extent on the size of the channel. If the rough projections, pebbles, etc., are large enough to produce cross-currents throughout the body of the stream, they will have much more effect than pebbles of the same size in a large bed. Small ditches have been built, using $n = 0.025$, and operated satisfactorily, while a few large canals have been constructed using the same value with the resulting velocity altogether too high. It would appear, therefore, that the value of n should not be governed by the character of the material in the bed only, but should be graduated according to the capacity as well.

Mr. Bishop.

LYMAN E. BISHOP, ASSOC. M. AM. SOC. C. E. (by letter).—Since writing the paper a set of velocity diagrams has been added to the yardage diagrams described.

As stated previously: In the yardage diagrams the dimensions of the sections alone govern the yardage quantities in any particular section diagram; the elements, n, s, and v, can be varied to suit the requirements of the engineer, and he can select the yardage diagram which most nearly meets the required conditions.

Briefly, the yardage diagrams had two defects: Each section was designed for a particular value of n, or coefficient of roughness; and the same velocity, 2.50 ft. per sec., was used throughout the set. There are a great many cases where the particular class of material through which a canal is to be located and the method by which it is to be constructed will warrant either a higher or lower velocity and a different coefficient of roughness than was used in the design of the various standard sections. The changing of the elements, n, s, and v, to secure a given capacity and velocity, and select the yardage diagram which would best fulfill the new conditions, required no small amount of "cut and try" computations. To overcome these two defects of the yardage diagrams and to facilitate the selection of the section which would most nearly fulfill the required conditions, the velocity diagrams were constructed.

Five different velocity diagrams were prepared, one for each of the following values of n: 0.020, 0.0225, 0.025, 0.030, and 0.035. These coefficients cover the extremes of the value of n commonly used in the design of earth canals, and vary according to the class of material, method of construction, quality of finish, and régime of the operating canal. Fig. 3 is the velocity diagram for $n = 0.0225$. Each of these

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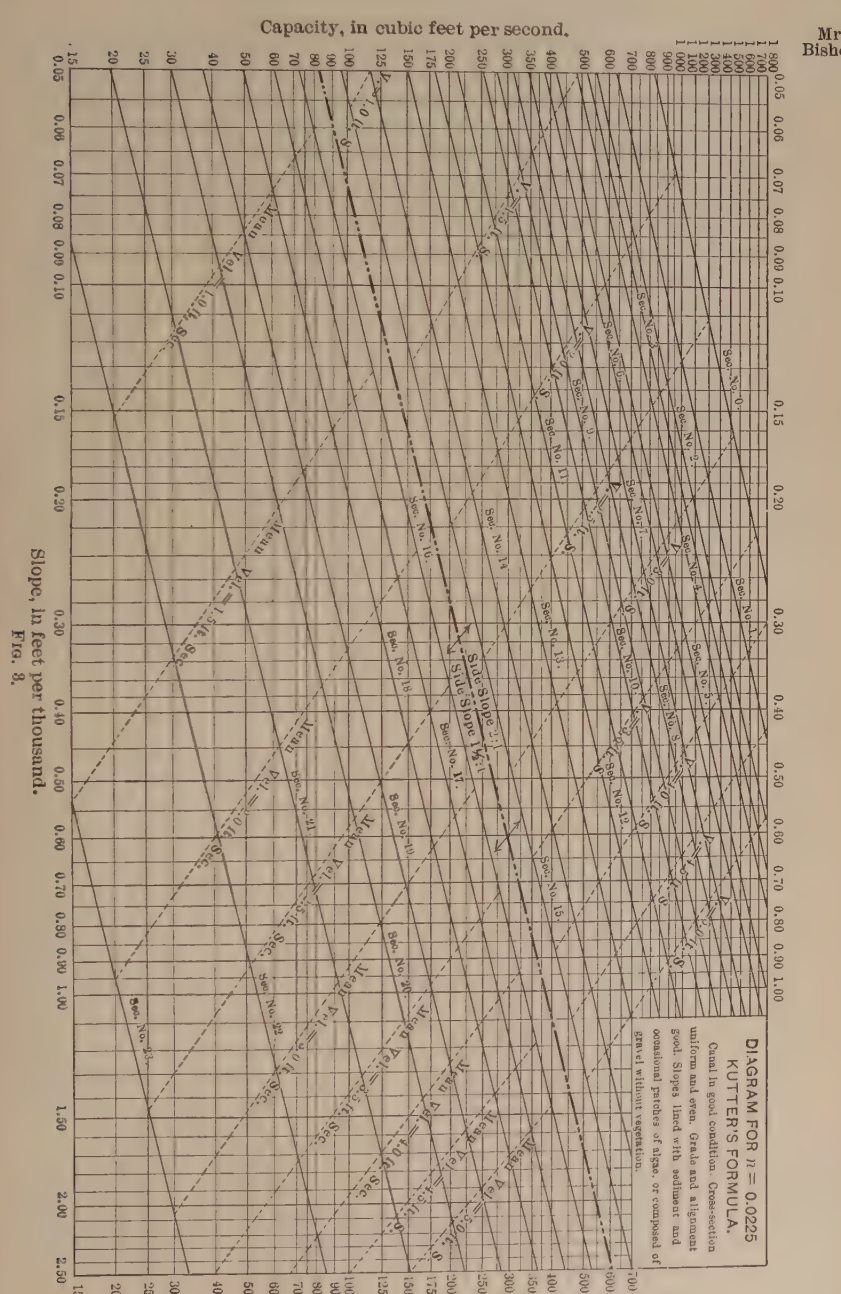
Capacity, in cubic feet per second.
DIAGRAM FOR n = 0.0225
KUTTER'S FORMULA.
Canal in good condition. Cross-section uniform and even. Grade and alignment good. Slopes lined with sediment and occasional patches of algæ, or composed of gravel without vegetation.
Side Slope 1 to 1
Side Slope 1½ to 1
Mean Vel. = 1.0 ft. Sec.
Mean Vel. = 1.5 ft. Sec.
Mean Vel. = 2.0 ft. Sec.
Mean Vel. = 2.5 ft. Sec.
Mean Vel. = 3.0 ft. Sec.
Mean Vel. = 3.5 ft. Sec.
Mean Vel. = 4.0 ft. Sec.
Mean Vel. = 4.5 ft. Sec.
Mean Vel. = 5.0 ft. Sec.
Slope, in feet per thousand.

Fig. 8.

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diagrams is on tracing linen, and is the same size (12 by 20 in.) as the yardage diagrams.

In the velocity diagrams, the section numbers of the yardage diagrams are plotted on a logarithmic scale; the vertical scale, or ordinates, being the capacity, in cubic feet per second; and the horizontal scale, or abscissas, the grade, in feet per thousand. The mean velocity, in cubic feet per second, was then plotted, using the section numbers of the yardage diagrams as one of the co-ordinate lines and the vertical scale as the other. As the capacity, grade, mean velocity, and the section number of the yardage diagram (which is a particular name used by the writer in this case for the hydraulic radius) are connected by an equation of the second degree, these quantities plot as straight lines on a logarithmic scale.

The heavy line divides the diagram into two parts: The water slopes of the section diagrams above the heavy line are 2:1, and those of the smaller sections below are $1\frac{1}{2}$:1.

Each of the velocity diagrams has the following properties:

(1) Each diagram is made for a different coefficient of roughness.
(2) Each diagram shows the different sections used in the set of yardage diagrams as straight lines.
(3) Each of these straight lines, a yardage diagram, gives the grade necessary to produce a given velocity and the capacity corresponding to this velocity.
(4) Each diagram covers velocities from 1.0 to 5.0 ft. per sec.
(5) Each diagram has a range of capacities from 1 800 to 15 cu. ft. per sec.

The velocity diagrams show the particular yardage diagram required to give a canal of any capacity with a mean velocity between 5.0 and 1.0 ft. per sec., and the grade necessary to produce this velocity and capacity with any selected coefficient of roughness.

For example, assume the following conditions: Coefficient of roughness, $n = 0.0225$; capacity, 400 cu. ft. per sec.; mean velocity required, 2.0 ft. per sec.

Enter the velocity diagram, Fig. 3, at the left with a capacity of 400 cu. ft. per sec., follow the 400 line horizontally to the right until it intersects the dotted line, $V = 2.0$ ft. per sec., which gives Section No. 10 as the proper one to be used; and vertically below is found the grade, 0.13 per 1 000, needed to produce the required capacity by using Section No. 10.

As a second example: Required, the capacity and mean velocity in Section No. 16, Fig. 2, if the value, $n = 0.0225$, is used, the grade remaining the same.

In Fig. 3 follow the vertical line, 0.45, until it intersects the diagonal line, Section No. 16. This intersection gives a mean velocity of 2.8 ft. per sec. and a capacity of 225 cu. ft. per sec.

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With any two elements given, the remaining elements of the required section diagram can be found graphically from the proper velocity diagram. The latter diagrams have proven a very efficient and valuable aid in the practical use of the set of yardage diagrams.

In replying to those who have discussed the paper, the writer desires to acknowledge their generally kind and courteous treatment. The discussions have been studied carefully.

The writer can hardly agree with Mr. Doolittle that the chief of the party should set the economic grade points. No special routine work should have to be done continuously by him. The levelman is the most important, and should have more practical experience than any other member of the party, except, of course, the locating engineer. Granting that the levelman has even ordinary intelligence, he sets the flags for economic cuts and also for short cuts (through ridges and saddles, and through fills across draws or gulches), and, where two alternate lines are flagged, he marks the estimate on the first flag of each line; then the country ahead is ready for the locating engineer "to study the ground," and grasp either one or ten stations as desired.

The writer usually carries an ordinary hand-level, and corrects any errors in judgment made by the levelman and changes the flags. Such errors usually affect only short distances, and occur on some part where either the fill or cut is in excess. After a week or ten days with the party, the levelman usually "falls in" with his work and, with a little assistance, soon learns what is wanted.

Where the country is fairly open, either uniform or rolling, and the P. T. is visible from the P. C., the majority of the transit points are made to fall on the station or the plus fifty. (Stakes are usually set every 50 ft.) This may be accomplished by selecting semi-tangent lengths slightly in excess of half the length of the curve desired. For example, suppose the locating engineer is at a desired P. I. in advance of the chainmen, and estimates the central angle as approximately 30°, and the length of curve needed, to swing into the next tangent ahead, as 250 ft. When the head chainman has advanced along the preceding tangent to within a distance, estimated by the chief as 125 ft., he puts in a hub at the nearest 50-ft. stake, back or ahead of where he is, at the discretion of the locating engineer. On chaining up to the P. I., this semi-tangent distance proves to be 140 ft. Two possible solutions of swinging into a (not the) desired tangent ahead, making the P. T. at an even stake, present themselves: The semi-tangent distance may be shortened, the delta lessened, and the new tangent may cross the original one at a small angle; or the semi-tangent may be lengthened, the delta increased, and, similarly, this tangent will cross the original desired one at a small angle. If neither of these solutions gives a desired tangent, the curve, of course, must be calculated, using the original semi-tangent distance.

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It will be assumed, for illustration, that in this case the P. I. is shifted back toward the P. C. and 130 ft. is estimated as the semi-tangent needed for 250 ft. of curve. The locating engineer, now at the new P. I., picks out the desired tangent ahead, and the chainmen measure the 130 ft. to the P. T. The transitman sets up at the P. C. and reads the angle, 16° 12′, to the approximate P. T. The following short computation is then made: $\frac{16° \ 12'}{5} \times 4 = 12° \ 56'$, which is the curve, and 250 ft. of this curve is then run in. The actual tangent distance for this curve and central angle is 128.7 ft., and the line obtained is parallel to the desired tangent and 0.7 ft. distant. Farther on (200 or 300 ft. or possibly even 50 ft.), another curve is run in, and the operation is repeated. By keeping odd-stationing down to a minimum, the chainmen make practically no errors, their work is easier, and they run more line in a day. The curves, of course, are of odd degree, but the odd calculating is done by the transitman, who is better fitted to do it.

This method will obviate calculating 80% of the curves on the line. When it is desirable to swing exactly into a long tangent ahead, the curve, of course, must be calculated. All curve data are usually computed in the field with an ordinary 10-in. slide-rule, and these are checked by the transitman with his slide-rule. A field handbook is always available, and, if the curve is sharp or long, the slide-rule computations can be checked with the tables. Tenths of feet can be read on the slide-rule for semi-tangent lengths and radii, and the nearest minute for the deflection or degree of curve. The actual length of the curve is computed in the ordinary manner by dividing the delta by the degree, and this is checked with the slide-rule. Mile after mile of canal has been located in this manner by the writer, and, in running in from twenty-five to thirty curves per day, probably only five or six are computed with the slide-rule, and possibly one or two with the tables. His experience on canal location indicates that about 300 ft. is the average length of curves and tangents.

Mr. Shipman brings up the question of "overhaul," that "bone of contention" between contractor and engineer. This trouble, as suggested, is found on irrigation work especially.

About three years ago the writer was resident engineer on the construction of 55 miles of a 300- to 250-sec-ft. canal, on which practically all the classification was earth. The head contractor sublet the excavation to twenty-nine sub-contractors at the following prices:

Earth excavation, 11 cents per cu. yd.;
Overhaul:

Free haul of 50 ft.;
Each 100 ft., 3 cents per cu. yd.

Mr. Bishop.

The head contractor received 16 cents per cu. yd. for excavation and 3 cents for overhaul, therefore the economic haul limit was 550 ft.

TABLE 2.—Overhaul Sheet.

Station.	Excess cut, in cubic yards.	Haul to station.	Cubic yards.	Computed overhaul.	Remarks.
316	172	314	112	168	Waste, 3 cu. yd.
		+ 50	30	30	
		315	22	11	
		+ 50	5	0	Station 316 + 50, waste 212 cu. yd.
317	239	318 + 50	7	7	
		319	42	63	
		+ 50	190	380	
+ 50	113	320	113	226	
318	62	320	41	62	Waste, 21 cu. yd.

As soon as the final cross-section quantities were available and the contractor had opened up a new piece of work, he was given a typewritten sheet showing the amount and direction of overhaul. Table 2 is a sample of such a sheet.

Station 316, for example, was defined as being from stake 315 + 50 to stake 316 + 00; and in every case the distance between the centers of gravity of excess cut and excess fill, was taken as the distance between the two stations.

This method worked admirably, and there was very little trouble with any of the contractors. An excess cut of 5 cu. yd. was often hauled 500 ft., under this system. Practically no borrow was made in the entire 55 miles. Where there would have been a borrow of any amount, the line was re-located and thrown farther into the hill. Then it was cross-sectioned again before the contractor was allowed to open up the work.

It might be interesting to Mr. Shipman and others to see a comparison between the overhaul computations and the final quantities on a 500-sec-ft. canal, located by the writer. The computations take in 70 stations of the line. The topography was fairly rough and broken. Overhaul was computed, using a mass diagram which was platted to a horizontal scale of 1 in. = 100 ft., and a vertical scale of 1 in. = 100 cu. yd. Earth excavation was estimated at 15 cents per cu. yd. The results are given in Table 3.

The lowest bid received on this piece of work was, earth excavation 14 cents per cu. yd., free haul 100 ft., and 1½ cents per cu. yd. per station overhaul.

Mr. Shipman raises an objection to making up monthly estimates from the transverse slopes and final profile. The writer has had a few "embarrassing explanations" to make in regard to cutting down the

Mr. Bishop.

yardage on a contractor's previous estimate. There are a great many cases where estimates have to be made up very carefully, yet the writer has had charge of construction work where the monthly estimates could be made up from the yardage diagrams. For example, 10% of the contractor's monthly estimate is held back until his final is turned in. The canal is only accepted in 5-mile completed sections. The contractor holds back 10% on all his "subs" until he gets his own final estimate. The contractor understands the method of obtaining his monthly estimates, and no explanations will be necessary. It is presumed, of course, that, as soon as cross-section quantities are available, they will be substituted for the location quantities.

TABLE 3.—Overhaul Computations.

Case No.	Length of free haul, in feet.	Overhaul: Total, in cubic yards.	Overhaul: Rate per cubic yard per station, in cents.	Total excavation, in cubic yards.	Total embankment, in cubic yards.	Waste, in cubic yards.	Borrow, in cubic yards.	Estimated cost.
1	200	11 456	1	34 012	33 356	650	0	$5 216.36
2	200	8 248	1½	34 012	33 356	890	230	5 260.02
3	100	10 794	1½	34 012	33 356	925	280	5 305.71
4	300	6 152	1½	34 012	33 356	760	95	5 208.33
5	100	14 332	1	34 012	33 356	645	0	5 245.12

The writer wishes to express his hearty approval of Mr. Porter's forceful and valuable discussion.

Somewhere between the two extremes of the air-line canal, which cuts or tunnels through the ridges, and fills or flumes across the gulches, and the balanced cut-and-fill canal, which "fits the ground" and follows the grade contour through its various twists and turns, a canal can be located which can be economically constructed and economically maintained. Probably every irrigation engineer has seen examples of both kinds, and the location mistakes on each are only too evident.

On one line located by an "old railroad man," the canal had a capacity of about 500 sec-ft., the transverse ground slopes varying from 5 to 20%, and the material excavated being a heavy, hard clay. Some of the tangents on this canal were 2 000 ft. long, the curves were all very light, and 800 ft. was a short curve. At practically no place did the canal fall below economic cutting. At the time it was constructed, the lower bank was built and the excess excavation was wasted on the upper side. In examining 19 miles of this canal, the writer estimated that an average of more than 300 cu. yd. per station had been wasted. The maintenance charges on such a canal will be exceedingly light, but economic construction was certainly disregarded.

Somewhere between these two extremes, is the "happy medium"

which the locating engineer must strive to find. He must ever bear in mind that his line must be economically located, that it is to be constructed, and—most important—that it must be maintained. If the locating engineer is not familiar with the principles of canal construction and operation, he is not worthy of his name, and his work will be a never-ending source of trouble. Mr. Bishop.

The writer failed to bring out one important use of the yardage diagrams, namely, that of avoiding numerous small curves in trying to follow the grade contour along a broken side-hill, and substituting a tangent or a light curve which will balance in excavation and embankment within the limits of economic haul. A tangent is picked out and flagged every 50 or 100 ft., levels and transverse slopes are obtained, and an estimate is made up from the particular diagram applicable. This estimate will show an excess of either cut or fill. Divide the assumed tangent into three or more equal lengths and obtain the average yardage per station and the average transverse ground slopes for each of these lengths. From the diagram, determine how far each of these sub-lengths must be shifted laterally to obtain the desired yardage. The ends of each of these sub-lengths are then flagged, the original flags are pulled up, and the balanced line is ready to fit with a tangent or easy curves. This property of the diagrams has been utilized frequently; the actual time taken in field work to balance up on a choppy side-hill is remarkably short.

The writer can hardly agree with Mr. Porter's broad statement that "curves are a constant source of trouble and expense." Sharp curves on a canal will always be the cause of heavy maintenance charges, but curves of a moderately long radius, will not be detrimental in the operation of any canal. The writer has seen long tangents on canals, where one bank was eroding and the other silting up and, within a short distance, the opposite bank was affected in the same way. Possibly this rather peculiar régime may be explained by considering the following points:

Uniform and mean velocity;
Uniformity of grade;
Trueness of excavated section for the water prism;
Capacity of canal—varied or practically constant;
Class of material through which the canal is constructed;
Amount, kind, and uniformity of silt in water carried.

Without going any further into detail, probably a consideration of these points, with respect to some troublesome curves, would at least divide the responsibility of their "waywardness" with their radius.

One additional point deserves some consideration in localities where there are high winds, as more wave action will be developed in the long tangents, and its eroding effect will be more pronounced on their banks than on those of the curves.

Mr. Bishop.

For every irrigation canal constructed in earth, there is some critical velocity which will just cease to erode the banks or deposit material held in suspension. This critical velocity is the potent factor in determining the limiting radius of curvature. What this relation is, the writer is unable to state, and would be glad to receive any information on this point.

Mr. Porter raises the question as to canals with one or two banks. Each particular piece of country has its own solution as to whether the canal should have one or two banks. If the upper banks are "washed out by nearly every local rain," then a very grave error has been made in their construction. If the drainage area above is large enough to deliver a sufficient quantity of water to wash out the upper bank, then a culvert or overshoot should be put in; or, the water should be taken into the canal, with sluice-gates and wasteways in the lower bank. In a two-bank canal, small drainage ditches above the main canal can often be constructed to good advantage. These ditches can pick up the super-drainage for $\frac{1}{4}$ mile on each side of a culvert, to which culvert they deliver their collected run-off. The yardage diagrams can be used for canals having one or two banks, as desired.

At present the writer is making a final location of a 350-sec-ft. canal in an exceedingly rough, mountainous and very heavily timbered country. For this work a topographic map, on a scale of 40 ft. to the inch, has been prepared. An accurately chained base line, approximately on the grade contour, is the "backbone" of the map. On this base line, topographic cross-sections were taken, using an ordinary transit and Philadelphia level rods. In the field office, the final located center line is projected on the map and this projection is retraced in the field. The yardage diagrams have proved very useful in locating the economic cutting points on the map. On this canal, the maximum curvature has a radius of six times the width of the water surface at full supply level, and this minimum radius is rigidly adhered to. Only the lower bank is to be constructed.

Mr. Williams suggests that the curves might be extended to cover transverse slopes greater than 35 per cent. The two yardage diagrams, Figs. 1 and 2, are from a series which the writer has called the "regular earth" set. In ordinary location, using an earth canal section, there are very few slopes greater than 35%; if steeper ones are met, the "regular earth" sections are not economical, and deeper, narrower ones should be designed to meet the conditions. The writer has more than eighty different yardage diagrams, some of which are designed for loose and solid rock canal sections, and covering transverse slopes as great as 65%; and yet, having all these different diagrams, there are cases where, on a special piece of location, a new canal section must be designed to meet new and different conditions.

Mr. Williams' comment on the value of Kutter's n (0.025), used in

Mr. Bishop.

computing Table 1, is well made. Tables are dangerous sometimes, particularly when their user goes by "rule of thumb," as was suggested. It might be mentioned again, however, that Table 1, includes velocities from 1.0 to 3.0 ft. per sec., and that, if a higher value of n is desired, the allowance can be provided for and the lateral section used which has the desired capacity and velocity with the selected value of n. In the design of a 10-sec-ft. canal, located in the same class of material and built with the same care as one having a capacity of 75 sec-ft., a higher value of n must certainly be used, if the two desired capacities are to be obtained.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1209

THE FLOW OF WATER IN CYLINDRICAL CONDUITS.

By Sidney A. Reeve, Esq.

This paper was suggested by a perusal (in 1909) of the paper entitled "Experiments at Detroit, Mich., on the Effects of Curvature upon the Flow of Water in Pipes,"* by Messrs. Williams, Hubbell, and Fenkell. It relates to that portion of the subject which treats of the distribution of velocities at various points in the cross-section of a pipe. Although the paper referred to is utilized as a text, the writer's purpose is to raise considerations which apply broadly to the general concept of the flow of water in pipes.

Such flow always occurs with velocities which are greatest near the axis of the pipe and least near its walls. Messrs. Williams, Hubbell, and Fenkell conclude that the distribution of these velocities is such as to be expressed, as a function of the distance from the axis, by an ellipse having the diameter of the pipe as its conjugate diameter. The velocity at the wall is said to be one-half that at the axis, while intermediate velocities vary as $(0.5 + y)\ V_c$, wherein V_c is the axial velocity and y is the abscissa of the ellipse, having its major axis equal to unity and its minor axis equal to the pipe diameter.†

Whether or not this be the true form of the function, there is no doubt that the velocities vary in some way generally similar to that

* *Transactions*, Am. Soc. C. E., Vol. XLVII, p. 1.
† *Transactions*, Am. Soc. C. E., Vol. XLVII, pp. 63 and 64.

stated. At the same time, the assumption seems to be general, among investigators of the subject, that the lines of flow are usually and naturally straight and parallel with the axis. The fundamental natural fact that water flowing in a conduit cannot proceed along straight parallel lines, under the basic laws of fluid equilibrium, and that it never does so, seems to have escaped from its proper place in the premises of their arguments, as well as from their pitometer observations.

All the filaments of water flowing side by side within a pipe must possess the same hydraulic energy. Energetic equilibrium within the pipe will always direct the flow of each particle in the direction of an equalization of energies, should any external influence upset the balance temporarily or locally. The filaments near the walls, however, are continually subject to such an external disturbing influence, in the form of the frictional resistance of the wall surfaces. Thereby they are constantly kept in a condition of deficit of velocity head. Thence results a continuous fault in the energetic equilibrium.

As the sum of pressure head and velocity head for all filaments must be equal, those near the walls, possessing lesser velocity head, must possess greater pressure head. This, however, is hydrostatically impossible; yet, hydrostatically, it would have to occur, if the filaments were moving in straight parallel lines, therefore the hydrostatic equilibrium becomes unstable, following the instability of the energetic equilibrium.

The water, in order to regain its stability of equilibrium, is forced, by this unbalance of hydrostatic pressure, to abandon its original—or, at least, its hypothetical—straight-line parallelism, and to turn to right or left into a helical path. Since the instability of equilibrium is indifferent as to right or left, in whichever direction it happens to be started it persists insistently, because it finds therein stability of equilibrium.

A similar phenomenon is familiar in the helical outflow of water from a stationary wash-bowl. While the source of the instability in this case—the radially centripetal lines of approach toward the outlet—is somewhat different from the case of frictional flow within a long cylindrical pipe, yet stability of equilibrium is regained in quite the same way; and frictional resistance is also present as a contributory factor, although of minor importance. It is virtually im-

possible to make the water flow from the wash-bowl in parallelism with the axis of the outlet. It insists on whirling helically to right or left, whichever way it happens to be started; and whichever way it whirls, it regains thereby its stability.

Within the helical whirl, the pressure head, acting radially outward, is not everywhere equal. Owing to centrifugal force, it is greater near the periphery. Equilibrium will be found when the rise in radial pressure, due to the greater radius and velocity of the whirl, as it passes from the axis to the wall of the pipe, just counterbalances the loss in velocity head due to the slower progress of the water along the pipe near the walls. The more the axial progress of the water is arrested by friction, the more it whirls, instead of progressing.

The energy of the whirl, however, cannot be all of that disappearing in the form of axial velocity. It must be something less than this, for the frictional resistance must be continually made good. The loss of axial velocity must go to two destinations, namely, the whirl, and the wall friction. The current parallel with the axis is both diverted and retarded by the wall friction. In other words, the oblique velocity along the helical path cannot be any greater, and must be somewhat less, than the maximum, axially-directed velocity in the center. Yet this oblique velocity is itself made up of two components, namely, (*a*) the velocity at any point in a direction parallel with the axis (that reported by the pitometer observations), and (*b*) the transverse velocity of whirl. That is, the helical velocity at any point in the pipe outside of the axis must be less than the velocity in the axis; and, because it is helical, instead of parallel with the axis, its axial component must be less, again, than its complete oblique self.

It is this second discount from the original maximum—the measurable axial velocity—which alone the pitometer measures in the traverse of the pipe away from the axis; but it is the first discount from the maximum velocity only, down to the oblique, helical velocity (only one component of which the pitometer measures), which supplies the energy visible in the whirl.

A mathematical solution of the situation will probably make it clearer. Let Fig. 1 represent the longitudinal section of a cylindrical conduit, having a diameter of 1 ft. Let V_c be the axial velocity therein, represented by the distance from the line, *aa*, to the point, *c*. Let V_w be the velocity, parallel to the axis, of the filaments touching

the walls, represented by the distance from the line, *aa*, to the line, *bd*. For the present it is sufficient to assume, as determined by observation, that $V_w = \frac{1}{2} V_c$.*

Let V be the velocity parallel with the axis at any point, m, in the cross-section of the pipe, at a distance, y, from the axis. Then at every such point the true velocity of the water will be a helical one, U, greater than V, because including some portion of the tangential

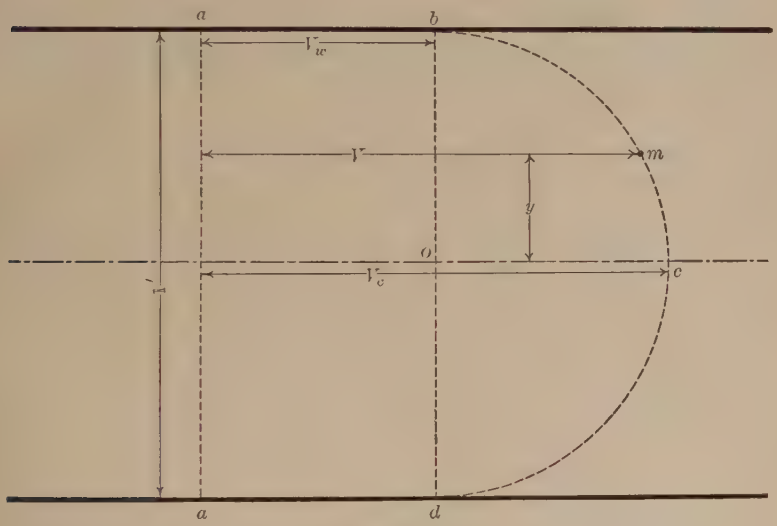


FIG. 1.

component, yet less than V_c, because constantly contributing energy toward the wall. It is the deficit in velocity head of U below V_c which must be made up in centrifugal force, in order to counterbalance the excessive pressure head near the wall; yet this centrifugal force is the result of a tangential velocity which is itself a component of U.

Let v be the tangential velocity at any point. Then

$$U^2 = V^2 + v^2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

Therefore, the deficit in linear velocity head which creates the deficit in pressure head necessarily counterbalanced by centrifugal force, is

$$\frac{1}{2g}(V_c^2 - U^2) = \frac{1}{2g}(V_c^2 - V^2 - v^2) \ldots\ldots\ldots\ldots (2)$$

* *Transactions*, Am. Soc. C. E., Vol. XLVII, pp. 63 and 64.

The centrifugal force which must be developed to counterbalance this, if stated in the form of a differential increase as one passes from the axis outwardly, is

$$dP_c = \frac{v^2}{y}\,dM = \frac{v^2}{y}\frac{dW}{g} = 2\,\frac{v^2}{2g}\,0.433\,\frac{dy}{y} \quad \ldots\ldots\ldots (3)$$

wherein 0.433 is the weight of a prism of water 1 sq. in. in cross-section and 1 ft. long.

According to the paper already referred to, the curve, *bcd,* of Fig. 1 is normally an ellipse. If the horizontal scale for velocities be chosen so that V_c is represented by a length equal to the pipe diameter, the problem will be simplified further by making the curve, *bcd,* a semicircle. As the pipe diameter has already been assumed to be unity, this is tantamount to assuming that the axial velocity, $V_c = \sqrt{2g} = 8.02$ ft. per sec., and that all other axial velocities which might occur in practice would beget phenomena everywhere proportional thereto—an assumption which seems to be justified by the facts now available. In that case,

$$\frac{V}{\sqrt{2g}} = \frac{1}{2} + \sqrt{\frac{1}{4} - y^2} \quad \ldots\ldots\ldots (4)$$

$$\frac{V^2}{2g} = \frac{1}{2} + \sqrt{\frac{1}{4} - y^2} - y^2 \quad \ldots\ldots\ldots (5)$$

and, from Equation 2,

$$\frac{1}{2g}\,(V_c^2 - U^2) = \frac{1}{2} - \sqrt{\frac{1}{4} - y^2} + y^2 - \frac{v^2}{2g} \quad \ldots\ldots\ldots (6)$$

It is the differential of this equation, converted from head into pressure per square inch by multiplication by the coefficient, 0.433, which must be equal to Equation 3.

From this final equation between Equations 3 and 6, we should be able to derive an equation for v or U in terms of y. The writer found difficulty in evaluating this equation, however, and as it is not one of general value, but is illustrative only of the assumed case of an elliptic form of traverse curve of velocities, its mathematical complexities were dodged by a recourse to graphical methods.

The results are shown in Fig. 2, in which AC represents the axis of the pipe and WW one of its walls. The abscissas of the curve, E, measured from AW, give the values for $\frac{V^2}{2g}$. Those of the curve, F, also measured from AW, give the values of $\frac{U^2}{2g}$. Those of the curve, F,

measured from CW, give the increment of centrifugal head over the axis, and the abscissas measured between the curves, E and F, give the values of $\frac{v^2}{2g}$.

Table 1, of values taken from Fig. 2, will also be of aid in realizing the situation. In this table α signifies the angle which a tangent to the helix at any point makes with a line parallel with the axis through the same point. Were the flow in any actual pipe quite in accord with the rigid theory of an elliptical traverse curve on which this

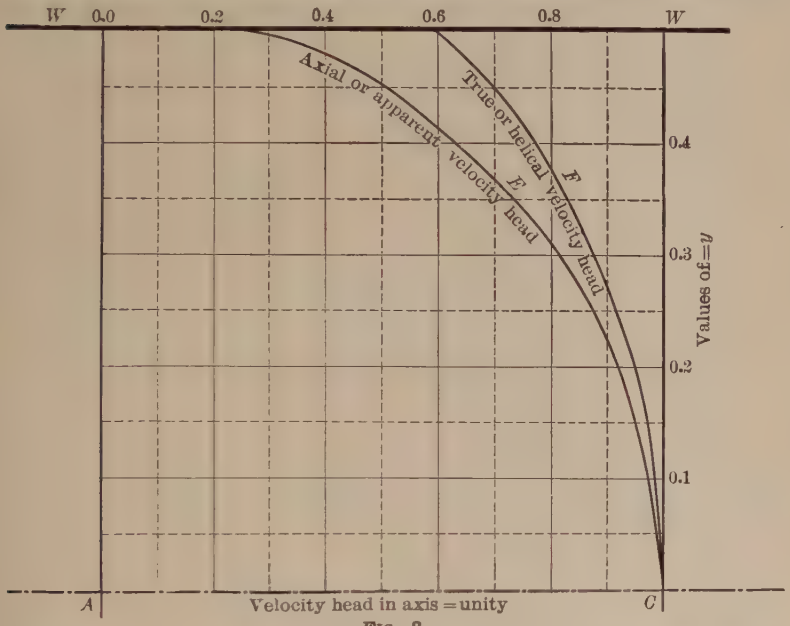


FIG. 2.

argument has proceeded, the angle, α, is the one at which the pitometer should be placed in order to obtain a maximum reading, for that particular distance from the axis of the pipe. Messrs. Williams, Hubbell, and Fenkell speak of noticing negative readings at times,* and of observing, in their investigation, maxima and minima at pitometer positions forming an angle with the axis; but the only angle mentioned is one of 45 degrees.

The writer, however, does not wish to give the impression that the actual flow is ever along truly helical paths. All that he is trying to

* *Transactions*, Am. Soc. C. E., Vol. XLVII, p. 56.

emphasize is that the condition of affairs within the pipe must be one of unstable equilibrium, as long as straight and parallel flow is imagined to occur. In reality, this is the one sort of flow which cannot imaginably occur. In avoiding it, however, the water is diverted, not into a smooth, true helix, but into the greatest intricacy of turbulent eddies.

TABLE 1.—Values Taken From Fig. 2.

y	=	0	0.10	0.20	0.30	0.40	0.45	0.48	0.49	0.50
$\frac{V}{\sqrt{2g}}$	=	1.000	0.990	0.958	0.900	0.800	0.718	0.640	0.600	0.500
$\frac{U}{\sqrt{2g}}$	=	1.000	0.993	0.972	0.934	0.875	0.834	0.803	0.790	0.774
$\frac{v}{\sqrt{2g}}$	=	0.000	0.077	0.158	0.249	0.354	0.425	0.480	0.515	0.592
$\frac{v}{y}$	=	.....	0.77	0.79	0.83	0.88	0.94	1.00	1.05	1.18
tan. α =		0.000	0.078	0.165	0.277	0.442	0.592	0.750	0.858	1.184
α	=	0	4° 27′	9° 22′	15° 29′	23° 51′	30° 38′	36° 52′	40° 38′	49° 49′

The fifth row of values, giving the ratio of the transverse velocities of whirl, v, to the distance from the axis, y, shows a fair degree of constancy. This signifies that the water might whirl, as it progresses, in a fairly homogeneous manner, not generally disturbed by the presence of the many small internal eddies just mentioned, although everywhere locally perturbed.

It will be noticed, too, that this ratio increases steadily and increasingly, as one passes from the axis to the wall. This is shown by Fig. 3. It is natural that this increase should occur, because there must be a continual flow of energy from the general body of water toward the wall; but it is not probable that the rate of increase rises as rapidly near the wall as is shown in Fig. 3, because the wall must have some retarding effect on rotational, as well as on axial, velocities. The form of curve shown is due to the fact that the geometric form of traverse curve specified by the writers quoted becomes tangential to the wall where it intersects the latter, the rate of its loss of velocity head then becoming instantaneously infinite. From all these considerations it appears probable to the writer that the traverse curve is not normally any particular geometric curve. It probably approaches the ellipse more nearly than any other form; but it probably loses its curvature somewhat, as it nears the wall, and intersects the latter at an appreciable, though slight, angle.

The entire situation will be best understood by keeping well in mind the distribution of energies in the mass of water. The mean axial kinetic energy of the water, of course, must remain constant, as long as the pipe diameter remains constant. This means that the supply of energy required for overcoming friction must be derived from a deficit in pressure energy, or head; and this is just what is observed in hydraulic phenomena, as a general fact. Friction reduces the pressure.

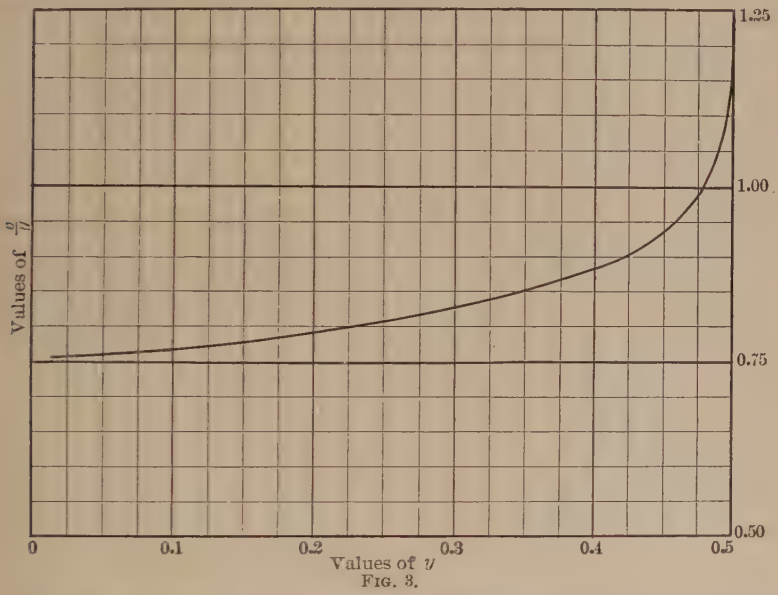


FIG. 3.

Strictly speaking, however, pressure energy cannot directly and of itself overcome friction. It is only motion which can do that. Friction is strictly a function of motion, appearing only when motion appears, and non-existent under the greatest pressures if motion be absent. It is merely a name for the retardation of motion by the contact of mass with mass. The familiar hydraulic rule that friction losses are to be deducted from head is a mere convention—of the utmost convenience, and unobjectionable for engineering purposes, but not strictly true—and hence misleading in such an enquiry as the present one, as to the true situation within the water in the pipe.

There is, therefore, a constant disappearance of motion energy along the walls in friction—for friction is overcome by motion

energy only as Arnold von Winkelried overcame the Austrian spears, by the extinction of the victor. This continual disappearance of motion energy along the walls, however, begets immediately, not the deficit of pressure head which it must finally become in form, after the books are balanced, but a local surplus of pressure head, due to the retardation of the water.

This surplus pressure can neither compress the water in the center of the pipe, nor (because it acts radially) hasten it along its way. All that it can do is to beget the instability of equilibrium which leads to the whirling action, or to the myriad of tiny eddies into which it breaks up, which has already been described. It is this general whirling motion about the axis which transforms, after the manner of a centrifugal pump, the needed deficit of pressure at its source or center, the axis of the pipe, into a surplus to meet the surplus pressure at the periphery.

There is thus effected a continual flow of energy from the general mass of the water to its periphery. Where the energy departs through the walls as heat; or, if it be still carried along with the water as heat, it is no longer hydraulic energy. The helical whirl in the pipe acts as a centrifugal pump for energy, using water as its working substance, yet discharging none of that water. It is only the energy which is discharged.

The situation may be better grasped by remembering that what is called the pressure of water is nothing but the centrifugal force of the tangential motion of its myriads of rapidly whirling molecules. It is quite natural, therefore, that it should be this form of energy which is called on to set up and maintain that general whirl of the entire mass of water which is needed to overcome friction at the expense of head.

The entire subject is much in need of investigation, in its arithmetical aspect, by the pitometer men. While such data as have been adduced here suffice to illustrate the general principles of action which must always be kept in mind, in conducting investigations of flow in pipes, yet it must not be supposed that they are reliable quantitative guides for situations other than the assumed premises of the argument.

One purpose of this paper, of educational rather than of practical importance, is to emphasize the fundamental mechanical fact that the natural path of mass in motion is always curvilinear. The straight

line is never a natural path of motion. It is always the natural path of force. Mass in motion not only never follows a straight line voluntarily, but it cannot be forced to follow it, except approximately and temporarily, under constraint, with constant and inevitable dissipation of energy. The only occurrence of the straight line in Nature is in the transmission of force through a solid; for neither liquids nor gases confine their transmission of force to straight lines. Indeed, the prime instance of the straight line in Nature—the foundation of all our astronomy and triangulation, and properly the sole basis for our definitions of geometric right lines—is the path of light through the luminiferous ether; and Lord Kelvin says that the luminiferous ether is a solid.

The secondary natural instance of the straight line, the transmission of natural force through solids, civil engineers learned long ago; their structural elements of roofs and bridges are never curved. It is only more recently and reluctantly that mechanical engineers, under pressure from such leaders as Professor Sweet, Mr. John Richards, and others, have learned to confine themselves more exclusively to straight lines.

The converse proposition, the curvilinear nature of all free motion, is thoroughly recognized in all the work of naval architects and hydraulic engineers; but our textbooks on elementary hydraulics, nevertheless, seem to have overlooked the importance of this fundamental fact. The principles of the flow of water in pipes is only a single instance of this.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1210

REINFORCED CONCRETE BRIDGE ACROSS THE ALMENDARES RIVER, HAVANA, CUBA.*

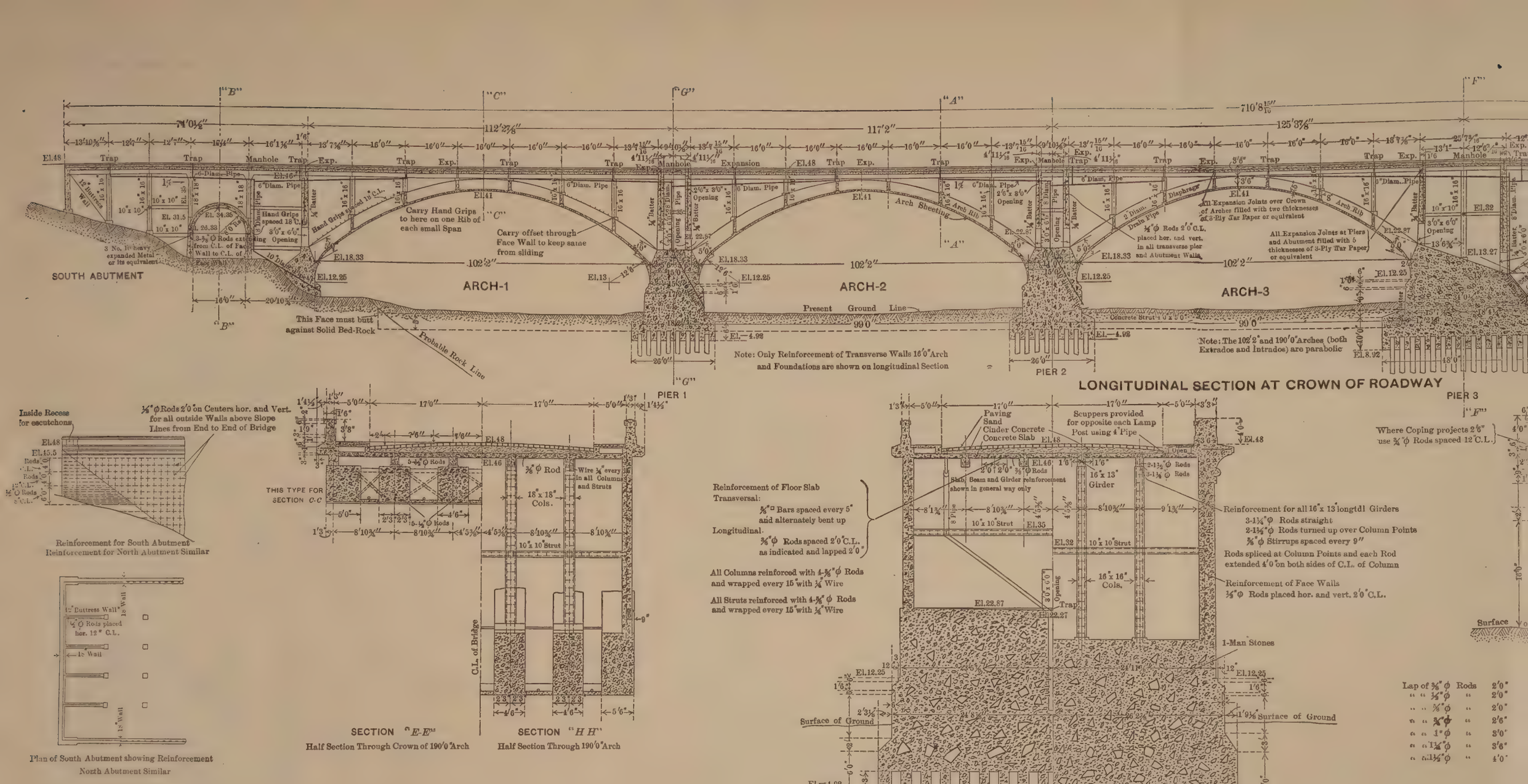
By Eugene Klapp and W. J. Douglas, Members, Am. Soc. C. E.

With Discussion by Messrs. L. J. Mensch, G. B. Strickler, and William Barclay Parsons.

The Almendares River Bridge (in the Province of Havana) crosses the Almendares River and connects the City with the suburb of Marianao. The construction of this concrete bridge was begun in November, 1908, and was completed in August, 1910. It is designed to carry highway traffic, and includes a double track for street cars. The live load was assumed as follows: On either car track, a concentrated load of 24 tons; on any other portion of the roadway, a wagon-load of 12 tons; on the sidewalks and for the arch design over the entire bridge, 100 lb. per sq. ft. of floor. The following are the principal dimensions: total length, 711 ft.; height, above the ordinary water level, 48 ft.; roadway, 24 ft. wide; each of two sidewalks, 5 ft. wide. The bridge is paved with granite blocks resting on 1 in. of paving sand.

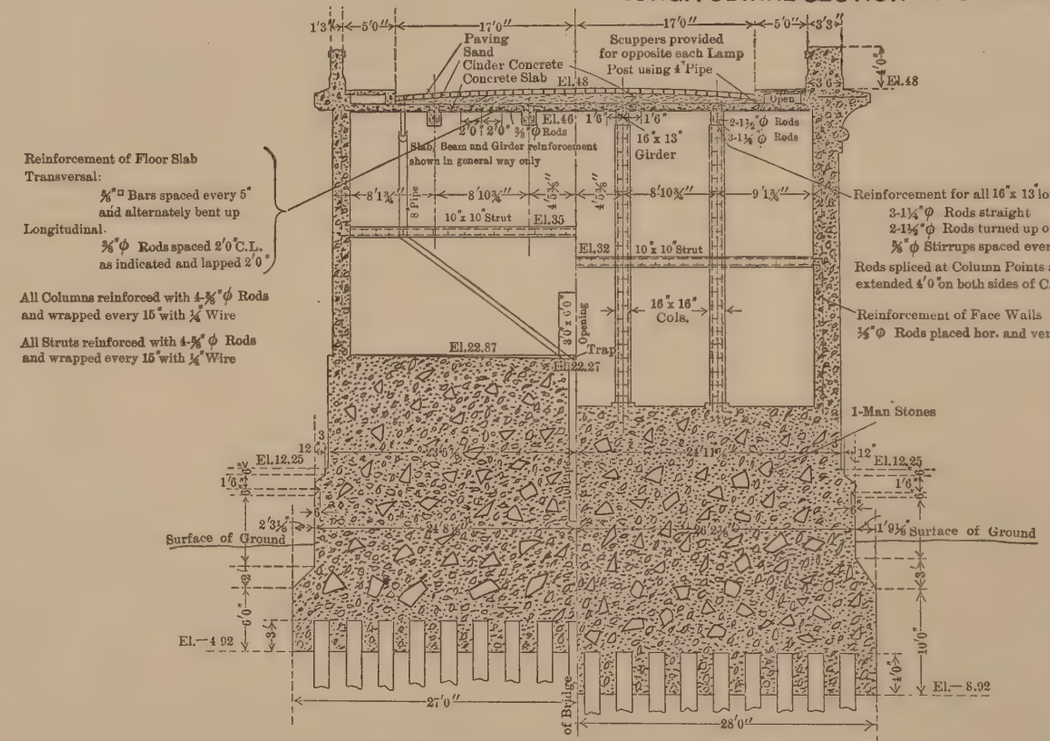
The structure consists of three spans of 102 ft. 2 in., having a rise of 22 ft. 8 in., and one span of 190 ft., having a rise of 32 ft. 6 in. The small arches are separated from each other by piers 15 ft. wide, and the large arch is separated from the adjacent small one by a pier 34 ft. wide, all measured at the springing lines.

* Presented at the meeting of November 1st, 1911.


"B"
"C"
"G"
"A"
"F"
710'8 15/16"
125'3⅜"
117'2"
112'2⅞"
74'0½"
SOUTH ABUTMENT
ARCH-1
ARCH-2
ARCH-3
PIER 1
PIER 2
PIER 3
Present Ground Line
Probable Rock Line
This Face must but against Solid Bed-Rock
Carry Hand Grips to here on one Rib of each small Span
Carry offset through Face Wall to keep same from sliding
Hand Grips spaced 18" C.L.
Arch Sheeting
Concrete Strut 1'0" x 2'0"
All Expansion Joints at Piers and Abutment filled with 5 thicknesses of 3-Ply Tar Paper or equivalent
All Expansion Joints over Crown of Arches filled with two thicknesses of 3-Ply Tar Paper or equivalent
Note: Only Reinforcement of Transverse Walls 16'0" Arch and Foundations are shown on longitudinal Section
Note: The 102'2" and 190'0" Arches (both Extrados and Intrados) are parabolic
LONGITUDINAL SECTION AT CROWN OF ROADWAY

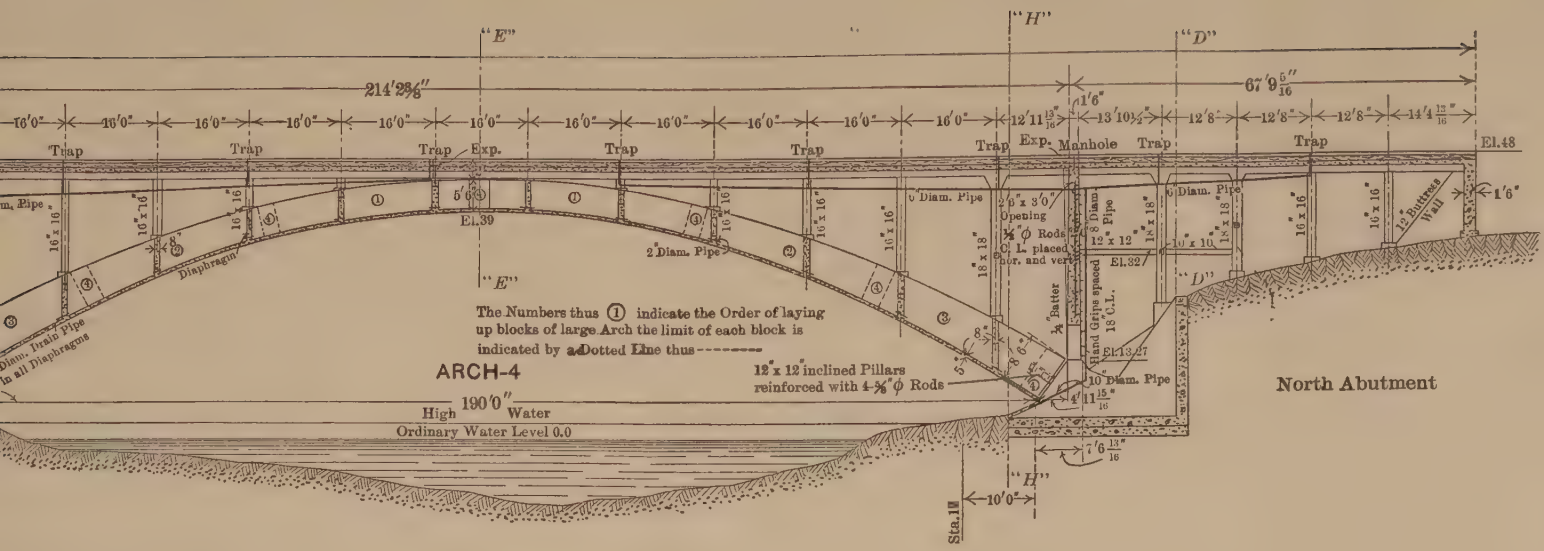
Inside Recess for escutchons
⅝"φ Rods 2'0" on Centers hor. and Vert. for all outside Walls above Slope Lines from End to End of Bridge
Reinforcement for South Abutment
Reinforcement for North Abutment Similar

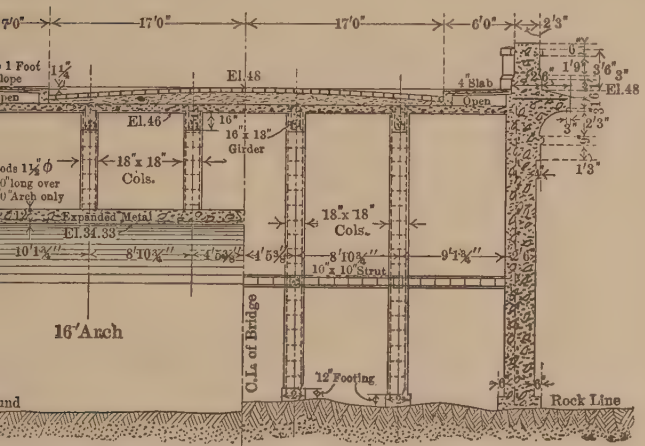
THIS TYPE FOR SECTION C-C
⅜"φ Rod
Wire ¼" every in all Columns and Struts
18" x 18" Cols.
10" x 10" Strut
C.L. of Bridge
SECTION "E-E"
Half Section Through Crown of 190'0" Arch
SECTION "H H"
Half Section Through 190'0" Arch


Paving
Sand
Cinder Concrete
Concrete Slab
Scuppers provided for opposite each Lamp Post using 4" Pipe
Slab, Beam and Girder reinforcement shown in general way only
16" x 13" Girder
10" x 10" Strut
16" x 16" Cols.
Reinforcement of Floor Slab
Transversal:
⅝"□ Bars spaced every 5" and alternately bent up
Longitudinal:
⅝"φ Rods spaced 2'0" C.L. as indicated and lapped 2'0"
All Columns reinforced with 4-⅝"φ Rods and wrapped every 15" with ¼" Wire
All Struts reinforced with 4-⅝"φ Rods and wrapped every 15" with ¼" Wire
Reinforcement for all 16" x 13" longtdl Girders
3-1¼"φ Rods straight
2-1½"φ Rods turned up over Column Points
⅝"φ Stirrups spaced every 9"
Rods spliced at Column Points and each Rod extended 4'0" on both sides of C.L. of Column
Reinforcement of Face Walls
½"φ Rods placed hor. and vert. 2'0" C.L.
1-Man Stones
Surface of Ground
C.L. of Bridge
SECTION "G-G"
Half Section Through Piers 1 and 2
SECTION "F F"
Half Section Through Pier 3

Where Coping projects 2'6" use ¾"φ Rods spaced 12" C.L.

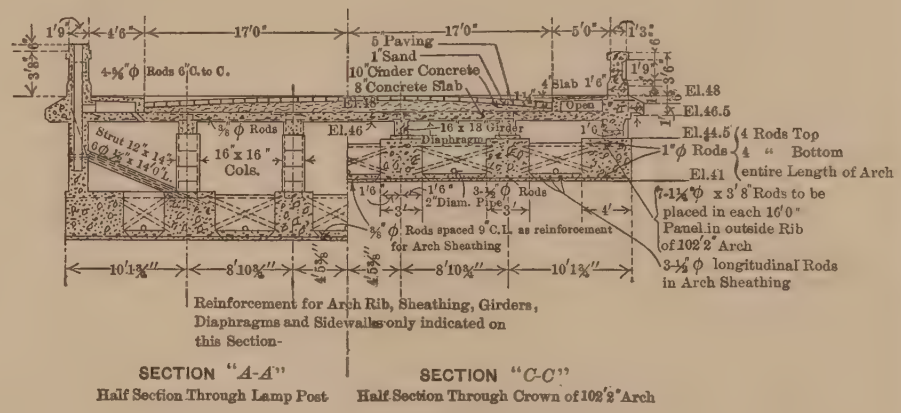
Lap of ⅜"φ	Rods	2'0"
" " ½"φ	"	2'0"
" " ⅝"φ	"	2'0"
" " ¾"φ	"	2'6"
" " 1"φ	"	3'0"
" " 1¼"φ	"	3'6"
" " 1½"φ	"	4'0"





SECTION "B-B"
ction Through South Abutment

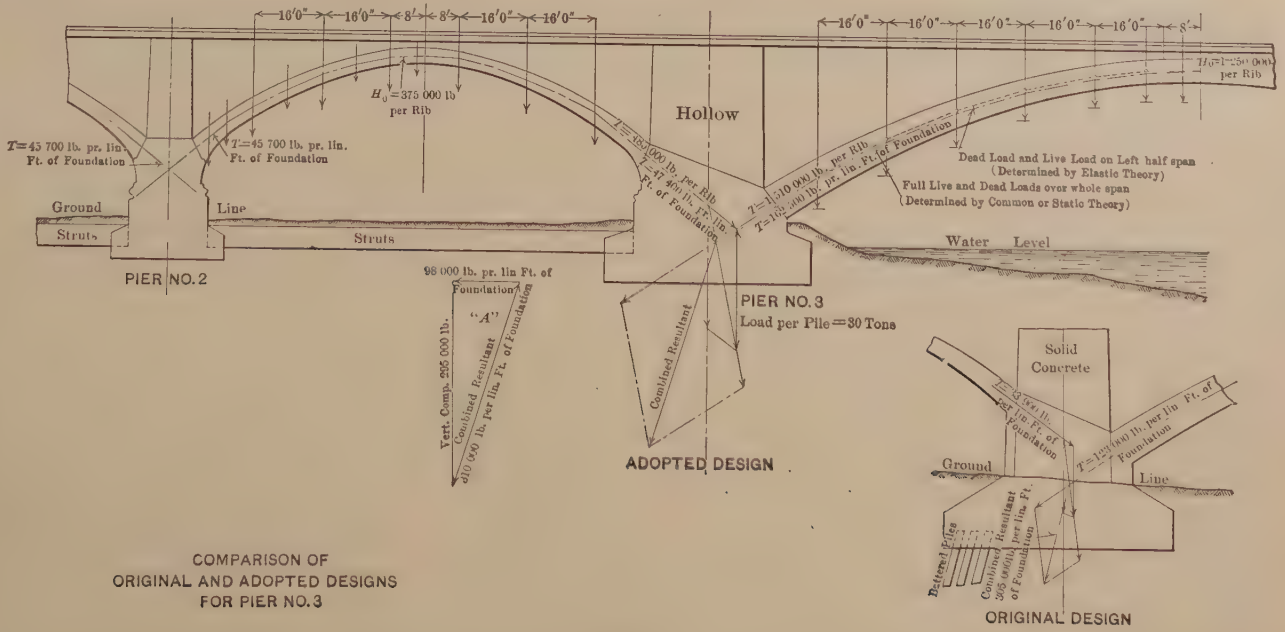
SECTION "D-D"
Half Section Through North Abutment



SECTION "A-A"
Half Section Through Lamp Post

SECTION "C-C"
Half Section Through Crown of 102'2" Arch

CONCRETE BRIDGE OVER ALMENDARES RIVER, HAVANA, CUBA.



COMPARISON OF ORIGINAL AND ADOPTED DESIGNS FOR PIER NO. 3

FIG. 1.

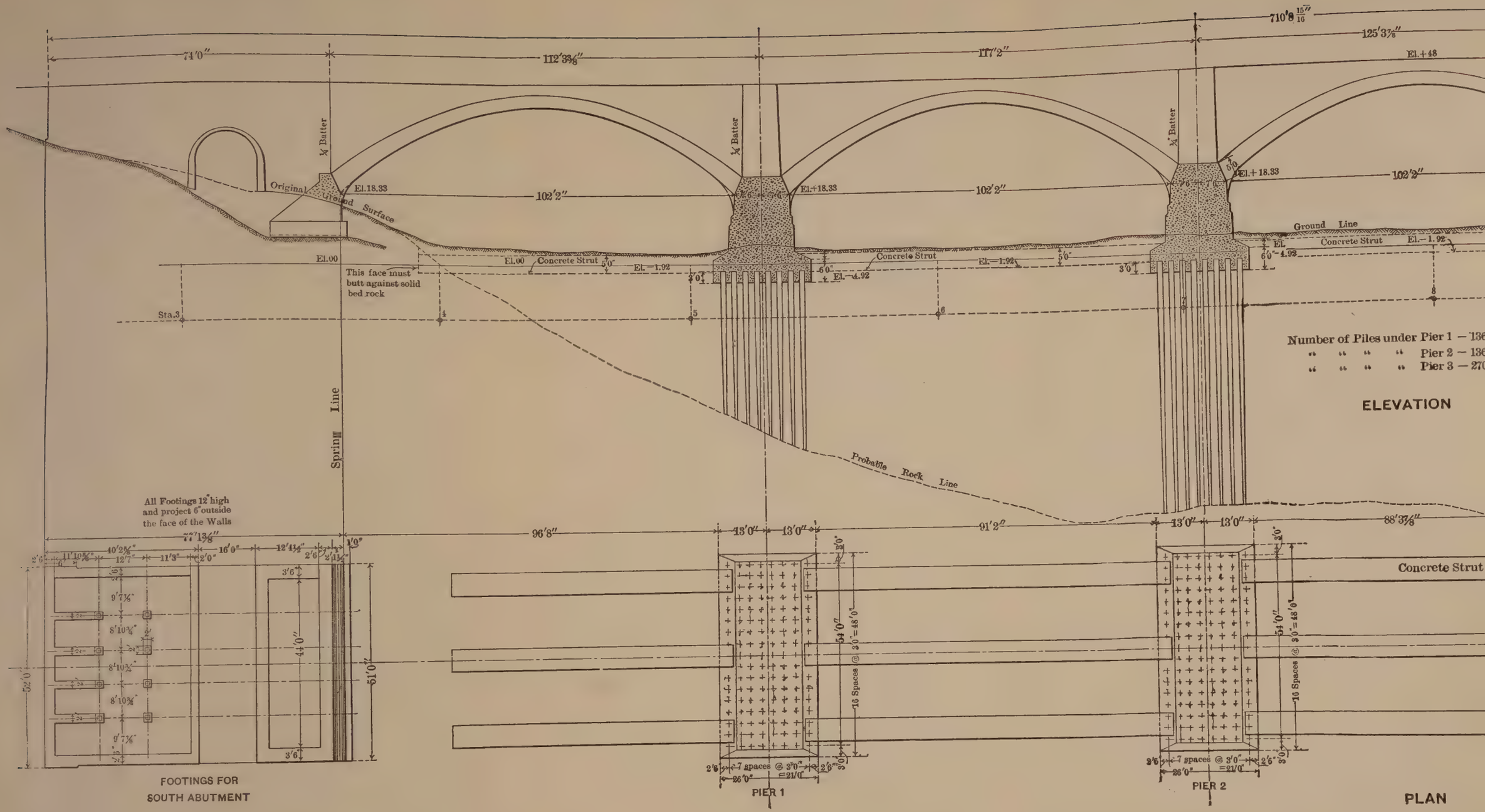
The ends of the bridge terminate in abutments approximately 70 ft. long, which appear to be of the U type, but were designed and built hollow, with interior buttresses to take care of the thrusts of the exterior earth.

All the piers rest on pile foundations driven to rock, the latter being of coral formation. The general design of these foundations is shown on Plates XXI and XXII. The arch abutments at either end of the bridge rest on coral rock. Hard rock was found at the south end, and a soft coral bed, which will be referred to in detail later, was found at the north end, where a solid foundation was much desired, in order to take the thrust of the large arch. The hollow abutment walls and buttresses rest on a firm, but rotten, coral rock, which is strong enough to take care of the light loads.

The contract drawings were made on the assumption that hardpan or solid rock would be encountered at a higher level than the subsequent test piles determined it to be. From the information obtained, the design for Piers Nos. 1 and 2 was changed from spread, reinforced concrete footings to pile footings, and Pier No. 3 was designed with vertical, instead of inclined, piles. In the original design, anticipating short piles, it was thought more economical to drive batter piles, as shown by Fig. 1, the inclination of the latter agreeing with the resultant thrust of the two arches resting on this pier.

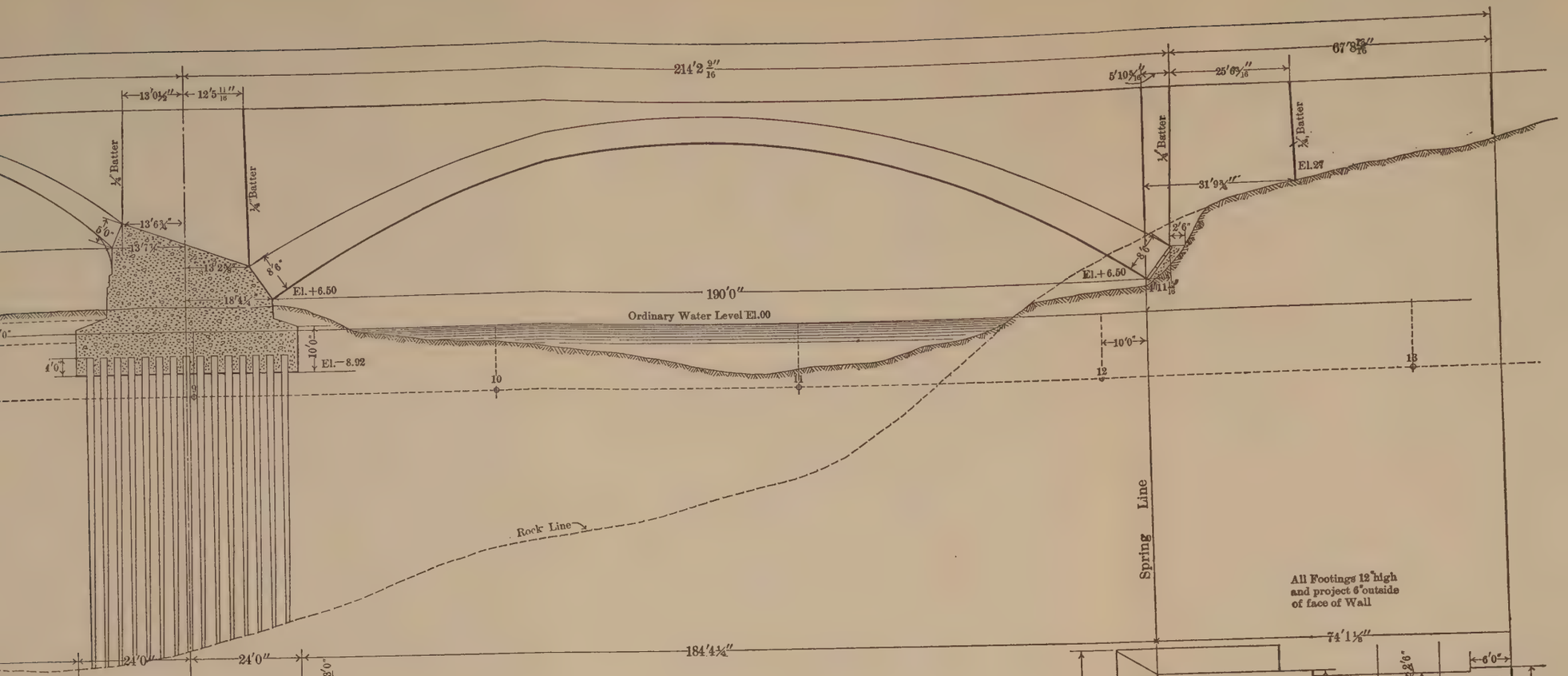
On account of the softness of the foundations, it was, of course, highly desirable to build a bridge of equal spans, in order to have vertical pressures on all piers and to get rid of the enormous unbalanced horizontal thrust on Pier No. 3. Local conditions, however, and especially the insistence of the Government, determined a span of 190 ft. over the river proper, and appearance prohibited the adoption of two flat arches of equal spans to cover the remainder of the distance between the south end of the large arch and the south abutment. Further, computations indicated that no material economy would be gained by using two approach arches in lieu of the three finally adopted.

It should be noted that, in the original design, Fig. 1, it was necessary to use a large mass of concrete above the springing lines of the arches resting on Pier No. 3, in order to force the resultant thrust to a more nearly vertical line. This load was undesirable because of the added cost and the increased load on the piles. To



710'8 15/16"
125'3 7/8"
74'0"
112'3 3/8"
117'2"
El.+48
¼ Batter
Original Ground Surface
El.18.33
102'2"
El.+18.33
Ground Line
Concrete Strut
El.00
El.−1.92
El.−4.92
This face must butt against solid bed rock
Sta.3
Number of Piles under Pier 1 — 136
" " " " Pier 2 — 136
" " " " Pier 3 — 270
ELEVATION
Spring Line
Probable Rock Line
All Footings 12" high and project 6" outside the face of the Walls
77'1 3/8"
96'8"
13'0"
91'2"
88'3 7/8"
Concrete Strut
16 Spaces @ 3'0" = 48'0"
7 spaces @ 3'0" = 21'0"
26'0"
PIER 1
PIER 2
FOOTINGS FOR SOUTH ABUTMENT
PLAN

TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXIV, No. 1210.
KLAPP AND DOUGLAS ON
REINFORCED CONCRETE BRIDGE.



FOUNDATION PLAN FOR CONCRETE BRIDGE OVER ALMENDARES RIVER, HAVANA, CUBA.

PIER 3

PLAN

FOOTINGS FOR NORTH ABUTMENT

decrease the leverage of the unbalanced horizontal thrust on the piles, the springing line of the large arch was made as low as possible and that of the adjacent small-span arches as high as possible. On account of the heavy loads brought on the foundations of Piers Nos. 1 and 2, it was not thought economical to increase further the thrust of Arch No. 3 on Pier No. 3, by building all the smaller arches of the retaining-wall and earth-fill type.

The piles of the foundations were designed to carry 30 tons each, and to be driven to such refusal as was necessary to carry this load, based on the *Engineering News* formula, in which the safe load is represented by the well-known expression, $S = \frac{2\,Wh}{s+1}$. However, as they were actually driven to rock, they could properly be considered as columns stiffened by the surrounding clay, but independent of skin friction for the transfer of the vertical load.

After test piles had been driven to develop the rock line, it was found that the rock, on which the piles of Pier No. 3 rest, was inclined with a sharp angle to the south, and that the inclination was in the same direction as that of the battered piles of the original design, and this fact, together with the great length of the piles, necessitated the abandonment of the original battered-pile plan.

At that time several other designs for this foundation were studied. A system of vertical piles with inclined caissons was considered, and also a foundation entirely of caissons. After a careful study of these types, it was finally decided to resolve the thrust into horizontal and vertical components, taking the latter component by piles driven vertically, while all the unbalanced horizontal thrust was taken care of by three lines of horizontal concrete struts extending from the south side of Pier No. 3 to, and beyond, Piers Nos. 1 and 2, to a firm rock resistance. These struts are shown on Plates XXI and XXII, and on Fig. 1, Plate XXIV. They were supported on piles driven at 10-ft. intervals. Although the ground was sufficiently firm to carry them, it was thought advisable, on account of the pier excavations at either end of each set of struts, to support them on separate pile foundations.

On account of their great length, the piles for the piers are of Georgia pine, the native Cuban piles being too short. Their average length is 56 ft. under Pier No. 1, 60 ft. under Pier No. 2, and 45 ft.

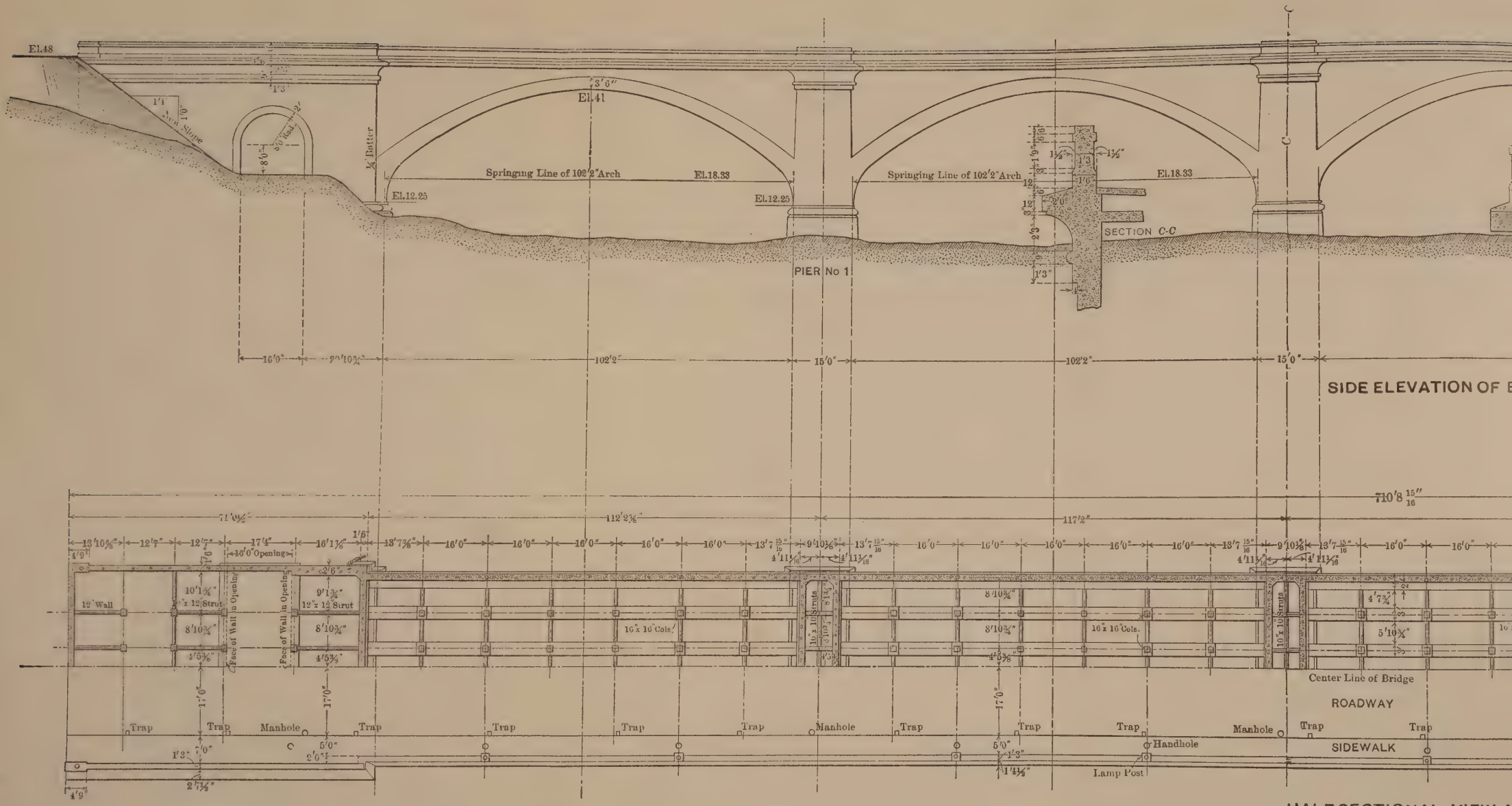
under Pier No. 3. The struts were built with construction joints between them and the abutting piers, in order to permit the piers to settle under their loads without cracking the struts. A layer of paper was placed between the face of each pier and the abutting struts, which made effective sliding joints. Pier No. 1 settled $\frac{3}{8}$ in., Pier No. 2, $\frac{3}{4}$ in., and Pier No. 3, $1\frac{1}{4}$ in., without cracking the struts, and apparently without causing much settlement of their ends.

On account of the softness of the foundation bed and the necessity of carrying the arches on pile foundations, particularly where a large unbalanced horizontal thrust had to be taken care of, it was of vital import to reduce the weight of the structure to a minimum consistent with stability. It was decided, therefore, to build the bridge with hollow spandrels and hollow piers above the springing lines, and to build the arches in ribs, carrying the roadway on a concrete slab supported by concrete columns resting on the back of the arch ribs.

The arch ribs were tied together by concrete diaphragms, in order to take care of the tendency of the ribs to buckle as columns. The slenderness ratio of the inner arch ribs of the large arch unbraced was 1:46, and that of the smaller arch 1:40. This columnar ratio was further reduced, so as to be negligible, by the solid arch sheeting, which was introduced in the design for this purpose as well as for appearance.

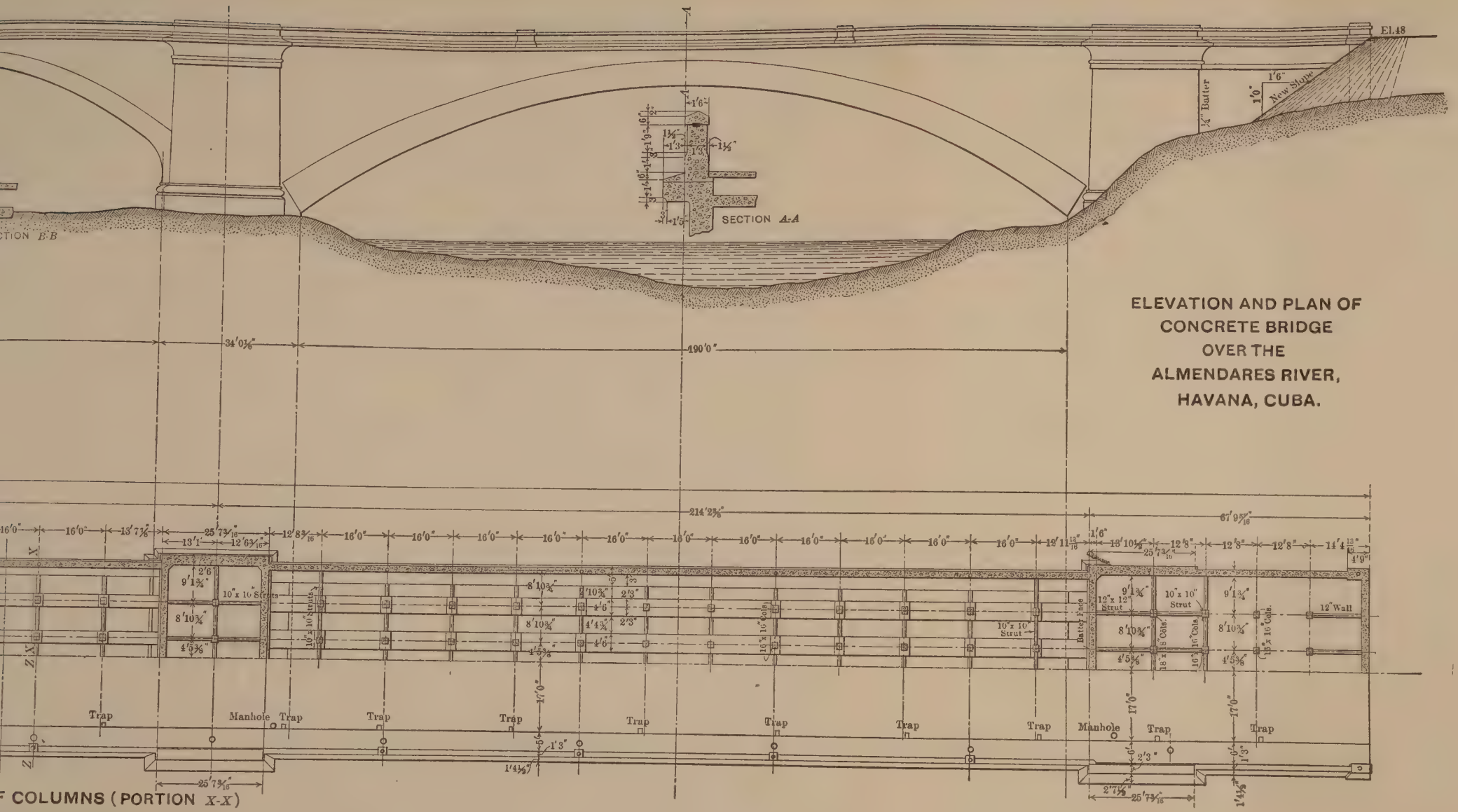
As one of the foundations of the large span was compressible, it was deemed necessary to build it as a three-hinged arch, converting it into a solid arch by grouting in the hinges on the completion of the structure. The rib type of construction is well adapted for hinged arches. In the case of the smaller arches, the deepened arch resulting from the use of rib construction, instead of a solid arch sheet, is best adapted to short spans carrying heavy concentrated loads. For a given volume of masonry, the line of pressure in a deep ribbed arch may depart a greater distance, without going outside the middle third of the ring, thus causing tension, than it could in an arch built with a solid or continuous sheet.

The detached rib design was further deemed desirable because the work was to be done at a considerable distance from the office of the designers. It gave, not only a better chance for inspection, but also a better opportunity to remove and replace any concrete of inferior quality which might by accident get into the work, and this seemed



HALF SECTIONAL VIEW A

HALF PLAN OF ROADWAY



ELEVATION AND PLAN OF
CONCRETE BRIDGE
OVER THE
ALMENDARES RIVER,
HAVANA, CUBA.

F COLUMNS (PORTION X-X)

EWALK (PORTION Z-Z)

to be an asset, although the work was to be executed under their supervision.

A bridge somewhat cheaper and perhaps quite as satisfactory in appearance might have been designed by omitting the sheeting between the consecutive ribs and also the solid spandrel walls shown on several of the drawings and photographs. However, as this bridge was to be built in a country in which the prevailing style of construction is of massive design, it was thought better, from an artistic standpoint, to enclose the column construction and detached ribs by curtain or spandrel walls and sheeting.

Having decided on a cellular type of construction, the proper curves for these several arches determined themselves. Since the funicular polygon, or linear arch, for an arch loaded uniformly, in its horizontal projection is a parabola, it was decided to adopt this curve for all arches. For appearances, however, the parabola at the springing line of the smaller arches was made tangent to the piers. The true parabola and the modified form are shown on Plate XXII.

The arches were designed by the common or static theory, it being assumed that the line of pressure lying generally nearest the neutral axis of the arch was the true one. After the thickness of the several arches had been determined, the line of pressure was computed and located by the elastic theory, checking very closely with the common or static theory.

Fig. 1 shows the lines of pressure in the large and small arches. The line of pressure shown in the small arch is for dead load, and the full line of pressure shown on the large arch is for full live and dead loads over the whole span. This line of pressure (and that for the small arch) was determined by the common or static theory, and the dotted line of pressure was determined by the elastic theory. The difference between the two lines for the large arch is shown to be almost negligible, and, as a matter of fact, the lines of pressure, as determined by the two theories for the same loading, were so close as to be practically coincident, and could not be shown on this small-scale drawing.

The line of pressure determined by the common method, not only for this bridge, but for other concrete bridges analyzed by the writers, agrees so closely with that determined by the elastic theory, that it is believed it can be used safely in design. However, in order

to make an independent check, the elastic theory would best be applied after the final curves of the arches and their thicknesses have been determined. Many engineers object to the elastic theory, because it is difficult to understand, and, when used for the first or second time, it is difficult and slow of application. However, if many arches are to be designed in an office, the elastic theory can be applied almost as quickly as the common theory, and requires less mental effort. The writers have found that the elastic theory may be applied by most draftsmen or computers without a great amount of study.

As this bridge was to be built in the tropics, the effect of temperature was hardly worth considering; but, in determining the maximum stresses, a range of 20° was allowed for. The writers are of the opinion that the result of considering temperature, in accordance with the elastic theory, is far from satisfactory, particularly where the arches are built in voussoirs or alternate blocks, or where the deformation of the centering will be so great that construction cracks will occur as the work is carried up. Further, unless the arches are heavily reinforced, they cannot develop the requisite amount of tension to make the structure act as an elastic arch under a change of temperature of 10 or 15°, depending, of course, on the rise and span of the arch.

Fig. 1 shows the resultant line of pressure on the foundations of the original design, the piles being driven parallel to the line of pressure, and each pile being designed for a pressure of 26 tons. In this design it was necessary to put in a heavy steel reinforcement for that portion of the foundation projecting beyond the neat lines of the pier proper, thus treating these projections as cantilevers. Fig. 1 also shows the design finally adopted for Pier No. 3, the piles being driven vertically to take the vertical component of 295 000 lb. per lin. ft. of foundation, and horizontal struts being introduced to take the horizontal thrust of 98 000 lb. per lin. ft.

The arches were designed so as to keep the line of pressure within the middle third, no steel being necessary to take any computable tension. Steel reinforcement was used, however, as an additional factor of safety, and was regarded as reducing the compressive stresses in the concrete, based on the moduli of elasticity of the steel and concrete.

The sizes of the ribs were determined by theory and on the assumption that no reduction in the allowable unit compression was

FIG. 1.—CONCRETE STRUT, 6 BY 6 FT.

FIG. 2.—NORTH ABUTMENT EXCAVATION, SHOWING CAVE HOLES IN BANK.

necessary on account of columnar action. The size and reinforcement of the transverse diaphragms and the thickness of the arch sheeting were matters of judgment rather than computation. The interior arch ribs support the reinforced concrete columns which, in turn, support the longitudinal beams or girders carrying the floor slab. In the original design longitudinal beams and transverse girders were used, but these were omitted in the final design, Plate XXI, in order to simplify construction and reduce the cost of form work, which, requiring experienced labor, is particularly expensive in the tropics. The longer columns on the piers were braced by horizontal struts, shown on Plates XXI and XXIII.

Several designs were made of the abutments, using gravity sections and buttressed reinforced concrete ones. These designs showed clearly that the hollow reinforced concrete type, the exterior walls acting as beams to carry the earth thrust and supported by interior buttresses, was materially cheaper. This type, shown on Plates XXI and XXIII, was adopted.

The architectural features of the original design were made extremely simple, on account of the lack of funds. Considerable attention, however, was given to the proper proportions of the various parts, the depth of the arch ring and the width of the piers being determined by the actual needs of the structure. After bids were taken, it was found that a small amount of ornamentation was permissible, and William M. Black, M. Am. Soc. C. E., Colonel, Corps of Engrs., U. S. A., then Engineer Advisor to the Government of Intervention, called into consultation Mr. T. S. Newton, an architect, to suggest modifications of the architectural details. Mr. Newton's suggestions were of such excellence that the designers gladly availed themselves of them, and modified the bridge design accordingly, materially improving its appearance. The writers believe such cooperation of architects with engineers to be far too infrequent.

Plate XXIII shows an elevation and plan of the bridge, and horizontal cross-sections at the tops of the columns.

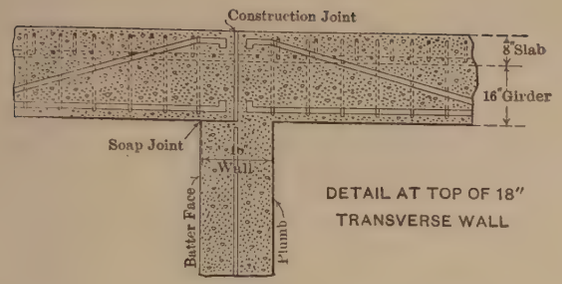
The outer ribs of the large span were reinforced with five 1-in., round rods extending the entire length, and on account of the eccentric load on this rib (see Section *E-E*), seven 1¼-in. rods, 4 ft. 8 in. long, were placed in each 16-ft. panel. The eccentricity on the outer ribs of the large arch amounted to 1 ft. 3 in.

Each diaphragm (Plate XXV) was reinforced with eight ¾-in. round rods extending well into the adjacent ribs. In addition, there were two ¾-in. rods at the top and bottom of the arch ribs, extending transversely across the bridge from side to side and through the diaphragms.

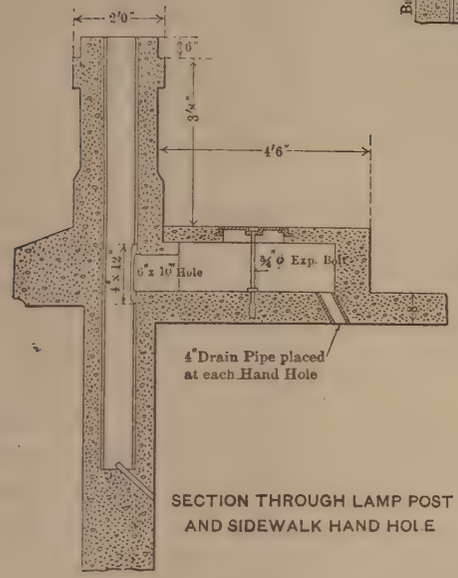
The arch sheeting, which was placed after the ribs had been concreted, was beveled, as shown on the transverse section through the arch rib (Plate XXV), so as to be independent of the ⅜-in. transverse rods, for retaining its position. The reinforcement of this sheet not only carries the dead-weight tension of the sheeting and any live load which may accidently come on it, but also forms, with the concrete, an efficient and continuous lateral bracing, the concrete composing the compression members, and the steel reinforcement the tension ones.

The reinforced concrete columns are doweled to the arch ribs by the column reinforcing rods, and the concentrated load brought by the columns is in part distributed over the arch ribs by the two ¾-in. round rods extending across the arch. The top of the columns is sufficiently bonded to the beams by the bending of the column rods, as shown on the transverse section on Plate XXV. The longitudinal beams were designed with the same amount of steel over the supports as at the middle of the beam, and were computed on the basis of $\frac{1}{12}\, w l^2$. Shear bars somewhat in excess of existing practice were provided, but, at the time the design was made, the question of shear or diagonal tension was even more indefinite than at present, and every precaution was taken to ensure the beams against such failure. The tops of the 18-in. columns were provided with knee braces to stiffen them, and these columns were increased in cross-sectional area on account of their greater length. The interior ribs of the large arch were reinforced with five ½-in. round rods near the intrados and extrados. The design of the smaller arches was similar to that of the large one.

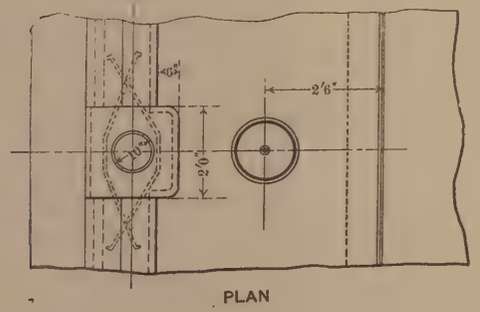
A very complete drainage of the roadway and the interior of the bridge was provided, as shown on Plates XXI and XXV. The drains were designed so as to carry the water off readily, and so that they could be cleaned without much trouble. The system of drainage is somewhat more expensive than that commonly used in the design of concrete bridges, but, particularly with a bridge of the hollow type, it

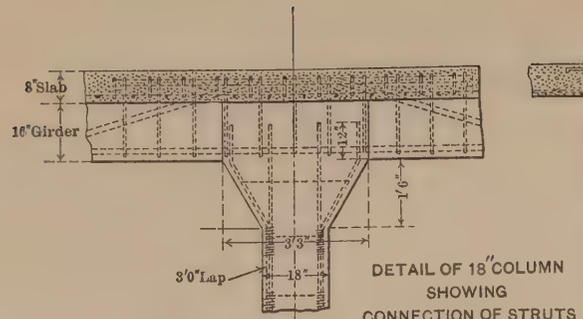


DETAIL AT TOP OF 18" TRANSVERSE WALL

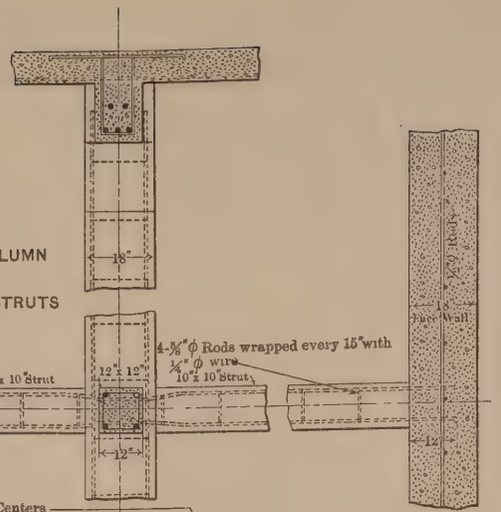


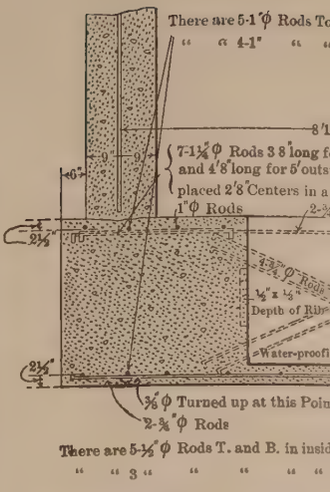
SECTION THROUGH LAMP POST AND SIDEWALK HAND HOLE



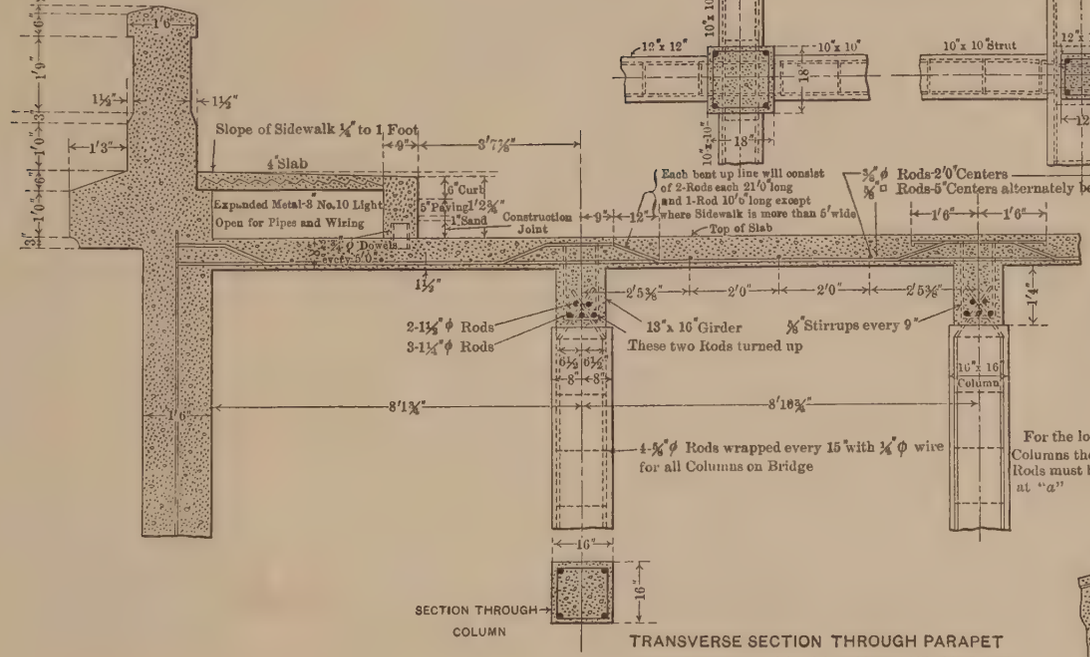


DETAIL OF 18" COLUMN SHOWING CONNECTION OF STRUTS

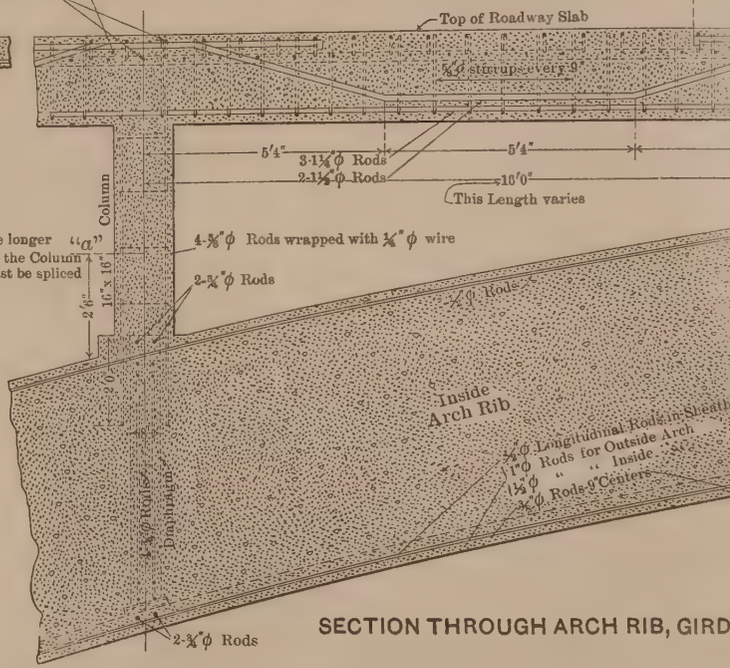



There are 5-1"ϕ Rods To
" " 4-1" " "
7-1¼"ϕ Rods 3 8" long f
and 4'8" long for 5' outs
placed 2'8" Centers in a
1"ϕ Rods
6"
2½"
¾"ϕ Turned up at this Poin
2-¾"ϕ Rods
There are 5-½"ϕ Rods T. and B. in inside
" " 3 " " " " "

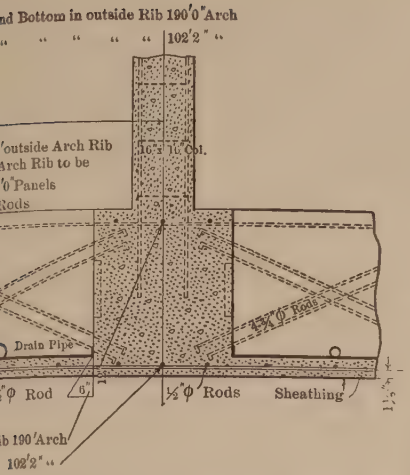
TRANSVERSE SECTION THROUGH A


Slope of Sidewalk ¼" to 1 Foot
4"Slab
Expanded Metal-3" No.10 Light
Open for Pipes and Wiring
6"Curb
5"Paving
1"Sand
Construction Joint
¾"ϕ Dowels every 7'0"
Each bent up line will consist of 2-Rods each 21'0" long and 1-Rod 10'6" long except where Sidewalk is more than 5' wide
Top of Slab
⅜"ϕ Rods-2'0"Centers
⅜"□ Rods-5"Centers alternately bent up
2-1⅛"ϕ Rods
3-1¼"ϕ Rods
13"x 16"Girder
These two Rods turned up
⅜"Stirrups every 9"
16" x 16" Column
4-⅝"ϕ Rods wrapped every 15"with ¼"ϕ wire for all Columns on Bridge
For the longer "a" Columns the Column Rods must be spliced at "a"
8'1¾"
8'10¾"
SECTION THROUGH COLUMN

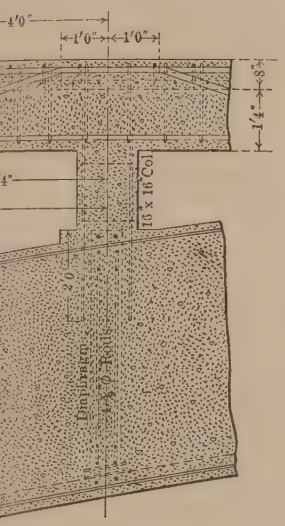
TRANSVERSE SECTION THROUGH PARAPET FLOOR SLABS, WALL, COLUMNS AND GIRDERS



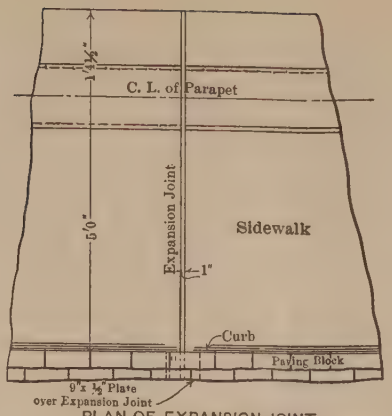
SECTION THROUGH ARCH RIB, GIRD



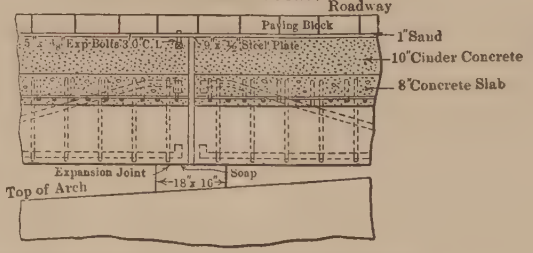
H RIB AND SHEATHING, SHOWING DIAPHRAGMS



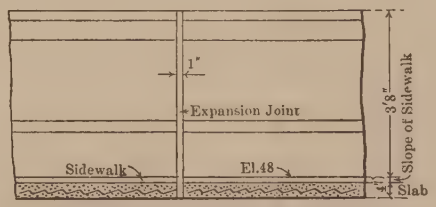
AND COLUMNS



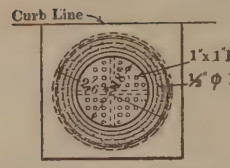
PLAN OF EXPANSION JOINT



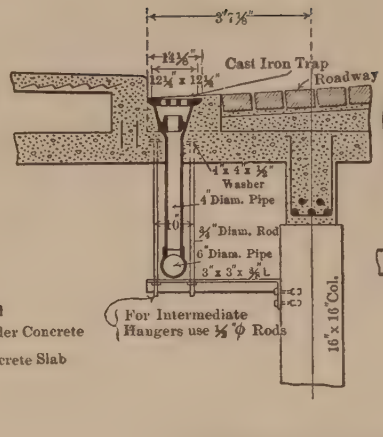
LONGITUDINAL SECTION AT CROWN OF ROADWAY

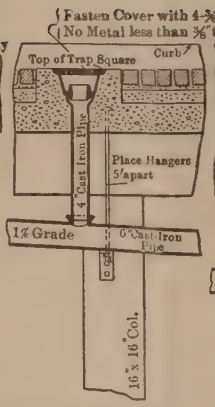


SECTIONAL ELEVATION THROUGH SIDEWALK
SHOWING PARAPET
DETAIL OF EXPANSION JOINT

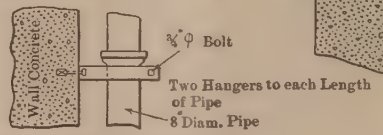


PLAN, CAST-IRON MANHOLE

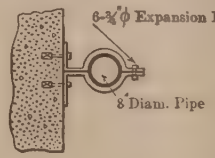


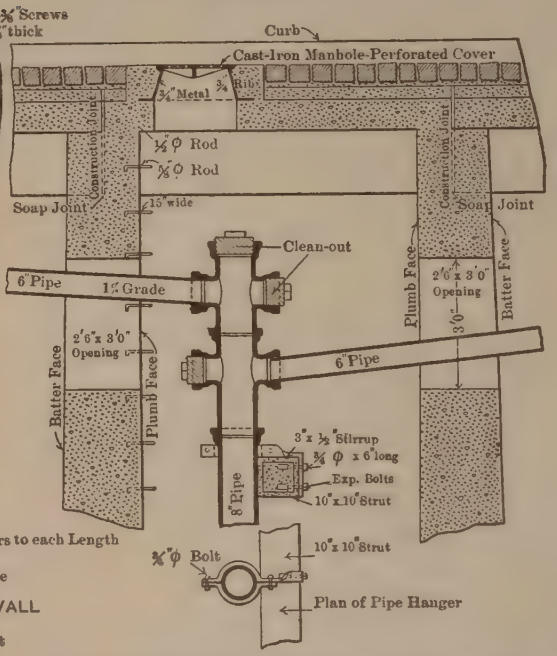


TRANSVERSE AND LONGITUDINAL SECTION
THROUGH TRAP



DETAIL OF CAST IRON PIPE FASTENER TO WALL





DETAIL SECTION AT MANHOLE-SHOWING DRAIN PIPES

DETAILS AND SECTIONS
CONCRETE BRIDGE
OVER
ALMENDARES RIVER,
HAVANA, CUBA.

was thought that the drainage should be efficiently taken care of in order to prevent any impounding of water within the curtain-walls.

The design of the abutments was made on the basis of an earth thrust of 12.5 h^2, and above the slope lines the walls were reinforced with ½-in. rods, 24 in. on centers, in both horizontal and vertical directions, in order to take care of the shrinkage cracks. Expansion joints at the piers were made as shown on Plate XXV.

In introducing the large horizontal struts to take care of the unbalanced horizontal thrust on Pier No. 3, it was foreseen that the thrust of the large arch would shorten the struts between Piers Nos. 3 and 2, and 2 and 1, and that this shortening would probably cause a rise in the crown of Arches Nos. 3 and 2. Measurements taken during the construction, as hereafter described, showed a material shortening of the struts, but did not show the rise of the arch crowns. This was probably due to the fact that the concrete, after it had set, lost the heat of setting, thereby tending to cause the arch ribs to shorten, and, further, the added loads of the columns and the floor-slab also tended to shorten the ribs and to lower the foundations, which lowering affected the arches to the same extent.

During extreme high water the water rises above the ground level between the smaller arches, and in a recent freshet, during a November cyclone, it rose to the height of the springing lines of the smaller arches. Though the current was sufficient to wash away the pontoon bridge adjacent to the site and otherwise damage the neighboring property, no scour occurred at these struts.

It was necessary to provide an opening through the south abutment sufficiently large for the passage of two teams abreast. This opening is shown on Plates XXI and XXIII. It has a span of 16 ft, and sufficient rise for the passage of vehicles.

The backs of all arches and intermediate sheetings are covered with two coats of coal-tar, in order to reduce unsightly seepage to a minimum, and drain pipes are provided, so that water may pass down the sheeting and be drained out near the springing line.

At various points along the bridge 10-in. wrought-iron pipes, about 9 ft. long, are set in the outside wall to provide for pockets in which lamp standards may be subsequently placed. These standards will be of sufficient size to serve for trolley poles if required. The details of the pipes and reinforcing steel are shown on Plate XXV, and their

locations are shown on the half plan of the roadway, Plate XXIII. In order to stiffen this somewhat thin outside wall against the kick of the lamp-post, if used as a trolley pole, the struts shown on Section *A-A*, Plate XXI, were introduced.

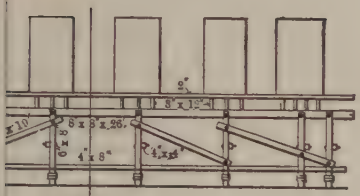
Calcareous sand was used in making the concrete for the bridge; this material is used in practically all Cuban work. Briquette tests, made from time to time, indicated that, when washed so as to be reasonably free from clay and mixed with Portland cement, the tensile strength is about 75% of that of similar briquettes made with standard Ottawa sand.

The sand was taken from sand-pits containing grains of varying coarseness and considerable clay. The washed sand contained not more than about 4% of clay, and showed a higher tensile strength than that containing about 7 per cent. The broken limestone used in the concrete was of varying hardness. It was difficult to obtain stone of proper hardness and reasonably free from clay. Compression tests on concrete containing small percentages of clay gave low results. Portland cement, manufactured by the Almendares Portland Cement Company, whose works are about 500 ft. from the south end of the bridge, was used in the construction of the bridge. This cement had been used successfully by the National Bureau of Public Works, and was accepted by the engineers of this bridge after passing the standard tests for American cements.

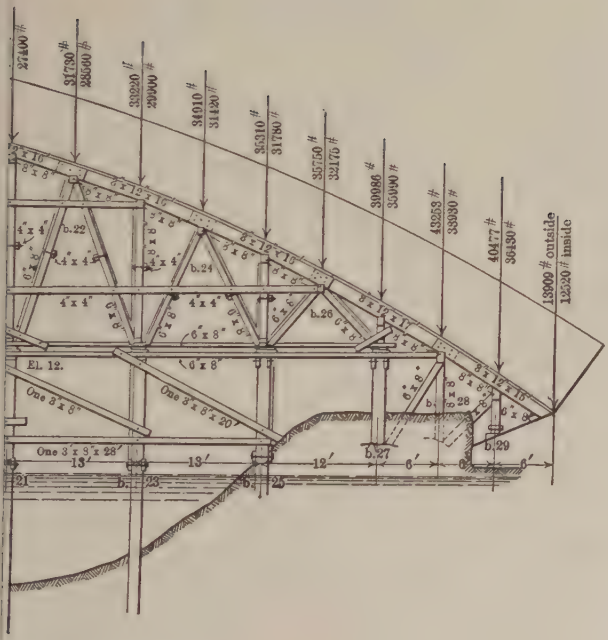
All steel reinforcement was purchased in the United States, and was tested before shipment to Cuba. The concrete was composed of 1 part cement and 7½ parts aggregate. The aggregate was composed of approximately 2½ parts of sand to 5 parts of broken stone, the exact relation of the sand to the broken stone being determined from time to time by the resident engineer in order to obtain a concrete of maximum density. The concrete was mixed in a Smith mixer of ½ cu. yd. capacity, and was conveyed from the mixer (beyond the south end of the bridge), by a cableway supported on fixed towers, to the point in the work where it was to be used. The materials for the concrete were hauled by teams and dumped from an elevated structure adjacent to the mixer.

Power saws were not used. Bends in the reinforcing steel sharper than 60° were made hot.

NTERING
RCHES OF
ES RIVER BRIDGE
ANA, CUBA



BENT 26



190-FOOT ARCHES

The contractor was required to carry up the exposed walls so as to bring the work to a horizontal joint at the close of each day, in order to make all the joints horizontal and regular. To ensure this result, the top surface of the day's work was tamped to a straight edge. In arch work the block joints were made radial so that the work of different days is shown by radial joints, care having been taken to build the arch bulkheads radially true and straight. An attempt was made to fasten the arch forms together by wire, but this resulted in somewhat unsightly work, with small and irregular bulges, and, in consequence, the ties were changed to $\frac{3}{4}$-in. bolts, designed so that their ends were removable after the work had been set up.

All exposed concrete faces were carefully spaded, so that there are no unsightly patches in the face work. Laborers in Cuba are not accustomed to this grade of concrete work, and it was found quite difficult to secure men to do the spading effectively, but after many attempts, two or three men were broken in, and satisfactory results were obtained. The bridge has not been tooled, except a small part of the inside portion of the parapet and the top of the hand-rail, but it is certain that dressing would not have added much, if anything, to its general appearance. This is partly due to the softness of the calcareous sand, which makes the concrete unsuitable for hammer work. After hammering the inside of the parapet, it was seen that the result was less satisfactory than rubbed work, and this latter method was finally adopted for the inside of the parapet and the top of the hand-rail. On work of this magnitude, the small form markings and occasional lipping streaks are not seriously objectionable, except for such portions of the parapet as are seen at close range.

The centering for the arches was designed by the Contractor's Engineer, T. T. Allard, M. Am. Soc. C. E., and approved by the Consulting Engineers. The design for the centering of the large arch is shown on Plate XXVI. Georgia pine lumber was used throughout, except for the wedges, where hard wood was used. The centering for the main arches was supported on piles, each pile carrying a load of approximately 30 tons. It was designed to carry only the dead weight of the arch rib. The centering for the smaller arches was supported in part by timbers resting on the concrete struts (which were used to take the unbalanced horizontal thrusts of the main arch).

These struts were not designed for this purpose, but were so used without showing any material settlement.

In the design of this centering, it was required that the wedges should be of straight grain. The allowable stresses in all timbers were computed on the basis of one-fifth of the reasonable breaking strain, the proper columnar deductions being made. The joists, caps, and lagging were computed to have a deformation of not more than ¼ in. under the full load of the concrete, the concrete being calculated as a liquid weighing 125 lb. per cu. ft. No dependence was placed on nails, except for fastening the lagging. The crown of the large center was raised 4 in. above the intended final height, and the crowns of the smaller arches were raised 1 in. above their intended height, in order to allow for settlement. The deformation and settlement of this falsework will be referred to later.

On the longitudinal section, Plate XXI, the small figures, included within the circles on the large arch ring, show the order in which it was intended that these arch rings should be built up. Work was to be started at the crown and then progress toward the springing lines, all keying blocks marked 4 to be placed on the same day. The keying blocks were made as narrow as possible while still permitting a man to work in them, so that after the main blocks had been placed (1, 2, and 3), these small blocks would not have sufficient weight to compress the centering a material amount.

Near the springing lines of this arch, two reinforced concrete pillars are shown, designed for supporting Block 3, it being intended that these pillars should be cast in advance of Block 3, and that, on the hardening of this block, the timber props, commonly used for supporting such a block, were to be removed, and this load was to be carried by the small concrete pillars. The main longitudinal reinforcement, in itself, would have been sufficient to carry the load, but the pillars were intended as an additional precaution. As a matter of fact, the angle of the soffit near the springing line is so flat that a block of concrete would not slide down the lagging of the center except under a severe shock. As described later, the method of constructing the large arch was somewhat modified, on account of the necessity of building it with hinges.

When the original design was made, it was thought that hard rock would be found for the foundations of the north abutment of the large

FIG. 1.—CENTERING FOR 102-FT. ARCH.

FIG. 2.—CENTERING FOR 190-FT. ARCH.

arch, but on digging down into this foundation bed a very inferior grade of coral rock, more or less decomposed, was encountered. Excavation was made to a depth of about 4 ft. below the natural surface of the ground, at the point marked "*X*" on Fig. 3 (also shown on Fig.

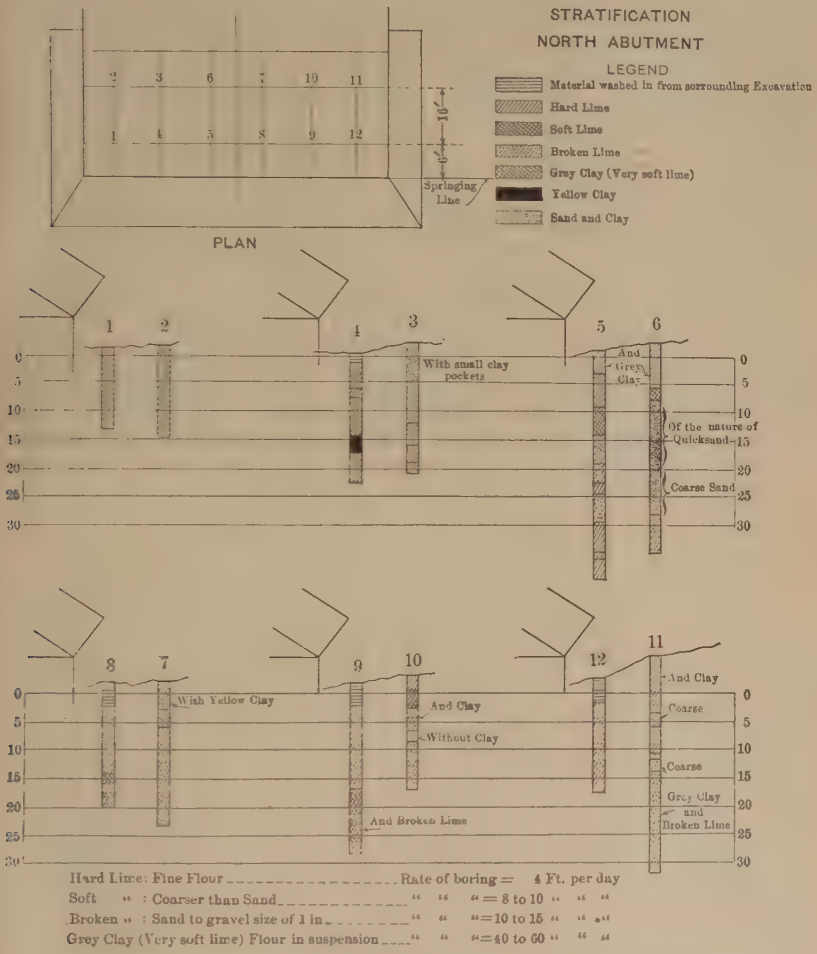

STRATIFICATION
NORTH ABUTMENT
LEGEND
Material washed in from surrounding Excavation
Hard Lime
Soft Lime
Broken Lime
Grey Clay (Very soft lime)
Yellow Clay
Sand and Clay
PLAN
Springing Line
With small clay pockets
And Grey Clay
Of the nature of Quicksand
Coarse Sand
With Yellow Clay
And Clay
Without Clay
And Broken Lime
Coarse
Grey Clay and Broken Lime
Hard Lime: Fine Flour — Rate of boring = 4 Ft. per day
Soft " : Coarser than Sand — " " " = 8 to 10 " " "
Broken " : Sand to gravel size of 1 in — " " " = 10 to 15 " " "
Grey Clay (Very soft lime) Flour in suspension — " " " = 40 to 60 " " "

FIG. 2.

2, Plate XXIV), and then was carried horizontally for about 34 ft. into the hill, forming a bank of decomposed coral rock at the back of the excavation. At that time some apprehension was caused by the apparent insecurity of the foundations for such a large span.

It was thought unwise to go any deeper, as two borings indicated that better foundations would not be encountered at any practical depth. Before proceeding with the final design of this abutment, twelve additional bore holes were sunk, as shown on Fig. 2, demonstrating that better foundations would not be obtained at any reasonably greater depth, there being at least 20 ft. of decomposed coral rock below the bottom of the existing excavation. These borings were a necessary precaution, as the surrounding coral hills show the rock to be full of caves varying from a few cubic feet to 20 ft. in width and more in length. The existence of the larger holes under the foundations of this abutment, and not covered by a sufficient thickness of rock, would have been disastrous.

On the basis of the borings, it was decided to design this foundation as shown on Fig. 3, in which the horizontal concrete pad was computed to carry all the vertical load, and a similar pad of concrete at the back to carry all the horizontal thrust, except a small amount which would be taken by the bottom pad due to the roughness of the foundation bed. The foundation was designed on the basis of a load of $2\frac{1}{2}$ tons per sq. ft. The exposed decomposed rock was sufficiently hard to have withstood a higher load per square foot, but the bore holes indicated the presence of small caves, from 2 to probably 12 cu. ft. in content, filled with clay, which was somewhat disconcerting. Similar cave holes had been encountered above the level of the footing, in the back part of the excavation, where the depth was about 16 ft. So many seams ran through this decomposed coral rock that small land crabs crawled through the bank without much difficulty.

On account of this foundation for the north abutment, and also due to the fact that the piles of Pier No. 3 averaged about 56 ft. in length, it was decided to build the large arch ribs as hinged arches, so that if the foundations settled the arch might deform without any initial strains or cracks.

Cast-steel hinges were out of the question, both on account of cost and appearance. Lead hinges were also too expensive. The hinges used consisted of cast-iron plates, 2 in. thick, covered with $\frac{1}{8}$-in. sheet lead, to provide a proper bearing for the hinges on the concrete above and below them. The surfaces of the adjacent concrete were dressed true to receive the hinges. On account of the introduc-

FIG. 1.—ARCH RIBS AND FORMS OF 190-FT. ARCH.

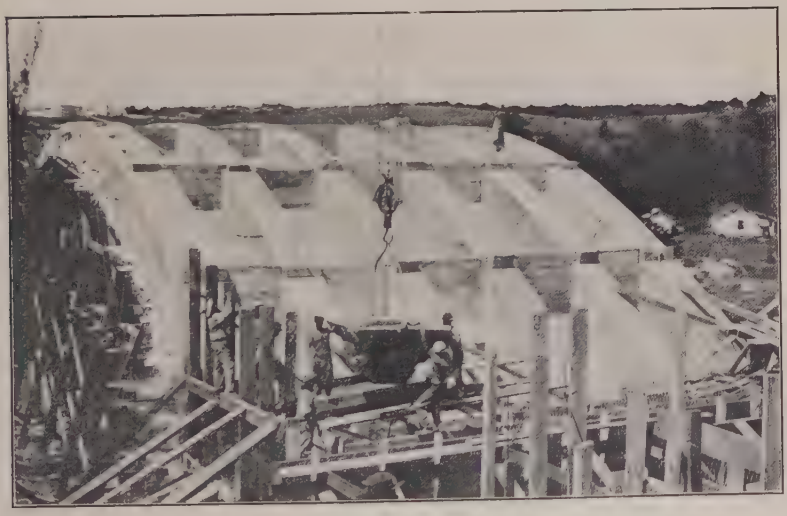


FIG. 2.—TOP VIEW OF 102-FT. ARCH.

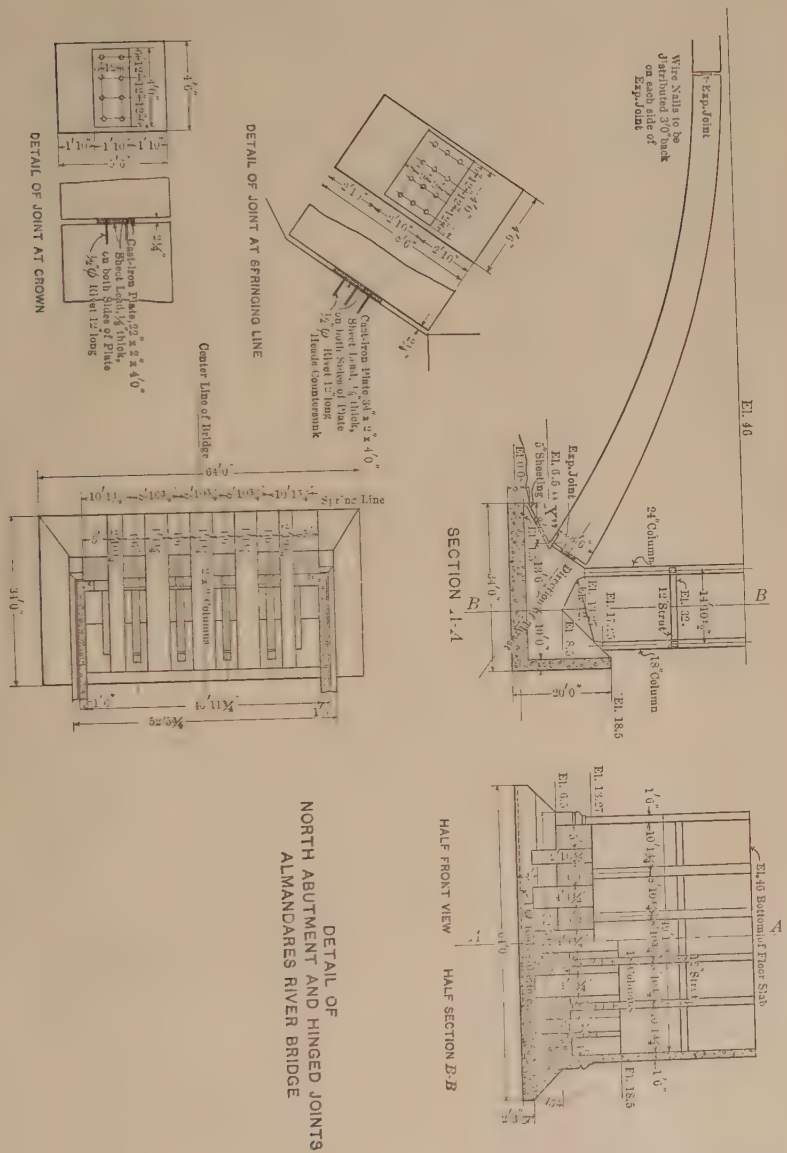

DETAIL OF JOINT AT CROWN
DETAIL OF JOINT AT SPRINGING LINE
Center Line of Bridge
Spring Line
PLAN
SECTION A-A
HALF FRONT VIEW
HALF SECTION B-B
Exp. Joint
Wire Nails to be J'stributed 3'0" back on each side of Exp. Joint
El. 46
24" Column
12" Strut
18" Column
El. 18.5
El. 46 Bottom of Floor Slab
DETAIL OF NORTH ABUTMENT AND HINGED JOINTS ALMANDARES RIVER BRIDGE

FIG. 8.

tion of the hinges, it was necessary to rearrange the size and order of laying the concrete blocks of the large arch. The order shown on Plate XXIX was followed, the lower large blocks being placed first instead of last, as originally shown.

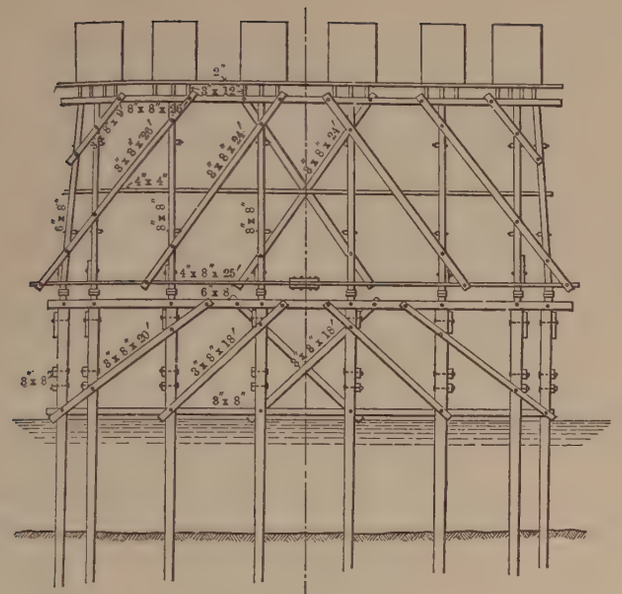
In building each of the smaller arches, the six ribs were first built monolithically over the central half of the span, followed by the haunch sections, completing one rib at a time. In the case of Arch No. 1, the sheeting and diaphragms were built after the completion of the ribs, and in Arches Nos. 2 and 3 the diaphragms were built at the same time as the ribs, and then were followed by the sheeting. Some poor concrete got into Arch No. 1, which made it necessary to cut out three sections and replace them.

Each of the ribs of Arch No. 4 was built in eight monolithic sections, and keyed up, two ribs at a time, from haunch to crown. The ribs adjacent to, and on either side of, the longitudinal center line, were laid first, followed by those immediately adjacent to these central ribs, and finally by the outside ribs. The sheeting and diaphragms were laid after the completion of the ribs. After the removal of the centering, it was noted that the concrete showed the most lipping near the oldest ribs, which would indicate that they had developed arch action prior to the placing of the sheeting concrete.

By referring to the centering of the large arch, it will be noted that the falsework was tied together by inclined bracing, so that if one post settled under its load it would tend to force down the posts connected with it by the inclined bracing, and, therefore, if the central ribs had been keyed and the ribs adjacent to it were built, these new rib loads would tend to strike the centering under the central ribs, and, by similar reasoning, the outside ribs would tend to strike the centering under the intermediate ones, and possibly under the central ribs also. On account of this action, it has been found expedient to build large arches in transverse blocks or voussoirs, rather than in longitudinal rings.

In arches of reinforced concrete, say, up to a span of 100 ft., the longitudinal method is quite practical, because of the slight deformation of the falsework. In the case of detached arch ribs, the building of these ribs independently is essential to the construction, and the striking of the ribs by the ones subsequently built is not objectionable, provided, however, that the concrete of the first ribs placed is suffi-

CENTERING FOR ARCHES OF ALMANDARES RIVER BRIDGE HAVANA, CUBA



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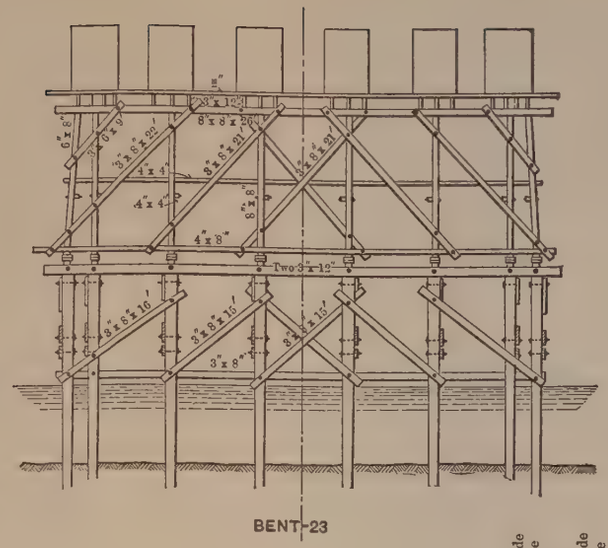

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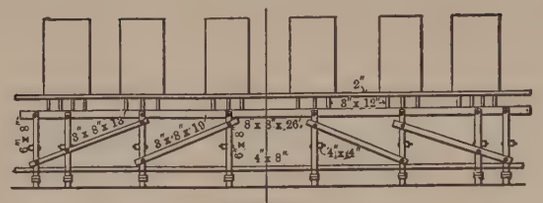

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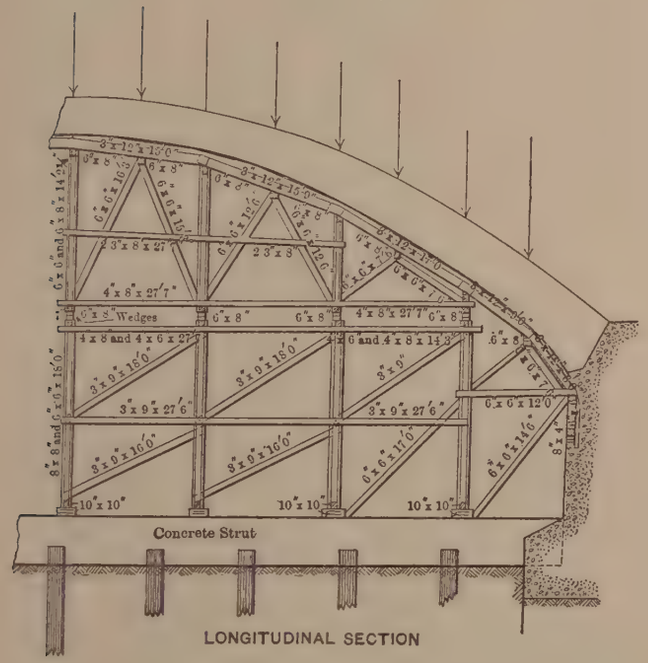
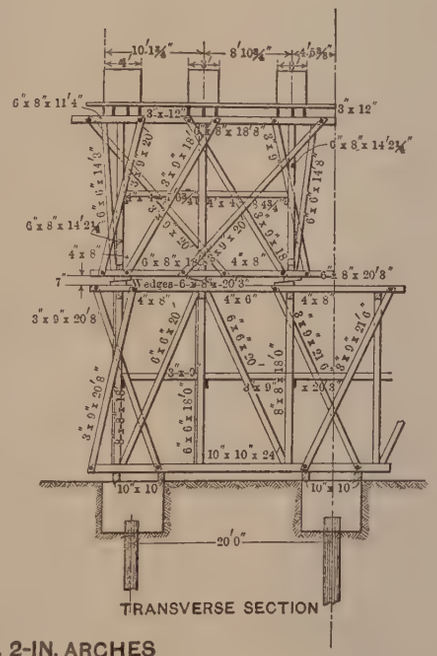
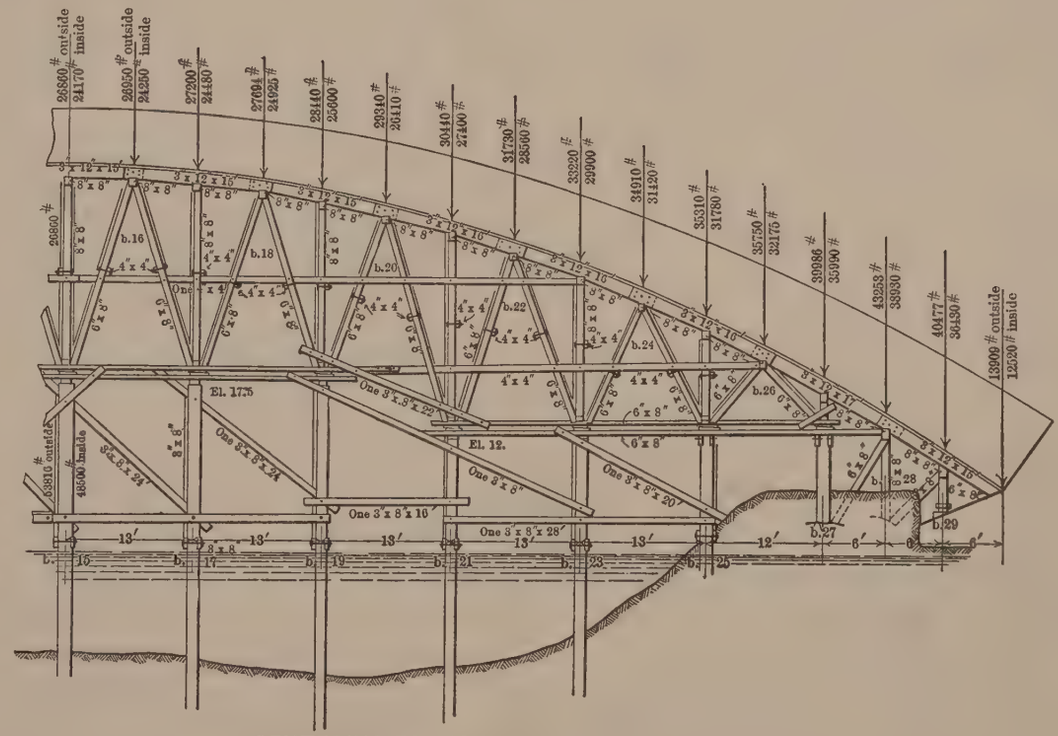


LONGITUDINAL SECTION



TRANSVERSE SECTION

CENTERING FOR 102 FT. 2-IN. ARCHES



CENTERING FOR 190-FOOT ARCHES

ciently set to permit of safely striking it. In some cases it might be well to see that columnar action had not developed to such extent as to buckle the first ribs, due to this inadvertent striking of the centers before the placing and setting up of the concrete diaphragms and sheeting.

Arch No. 1 was erected first, and then Arch No. 2, complete centering being furnished for both arches; then the falsework under Arch No. 1 was removed and set up for Arch No. 3. This was somewhat unprecedented, but it was done in order to save the contractor the expense of buying falsework for the third smaller arch. Care was taken, however, to watch Pier No. 1, which was founded on vertical piles, to see that there was no movement. Careful observation indicated that it did not move a measurable amount. This was partly due to the resistance of the concrete struts, which served to distribute the thrust of the arch throughout the piles of the entire system.

The falsework of Arches Nos. 2 and 3 was removed before the completion of Arch No. 4, similar observations being taken to make sure that there was no movement of Piers Nos. 2 and 3. It was planned to place vertical timber props under any arch from which the centering had been removed and adjacent to a pier showing movement, using the concrete struts as foundations to support the timber props. These props could have been placed in half a day, and they would have been just as efficient as the original centering.

Careful measurements, made at either end of the eighteen lead-covered hinge plates of the large arch, showed, upon striking, a change in the hinge opening of between $\frac{1}{32}$ and $\frac{1}{16}$ in. As there was no settlement of the abutment foundation, and only a reasonable settlement of Pier No. 3, the hinges were grouted and made solid prior to building the spandrel walls and columns. These hinges were designed so that the compressive stress in the concrete would not exceed 1 000 lb. per sq. in., even if the hinges were not grouted up until the bridge was complete. It was not intended to grout these hinges as long as there was any progressive settlement of the foundations. For concrete, this compressive stress, of 1 000 lb. per sq. in., is somewhat high, even when the area compressed is only one-third of the total area of the pressed surface, and therefore it was deemed advisable to reinforce with nails the concrete of the arches adjacent to the hinge and 2 ft. therefrom, measured along the axis of the arch. Tests made

on nail concrete by the resident engineer, although not entirely satisfactory, due to the small size of the cubes, clearly indicated its efficiency. This efficiency had been demonstrated previously by the Bridge Department of the City of New York.

TABLE 1.—STRUT SHORTENING.

Transverse Line "A" Crossing Struts.

Date	1-13-10	5-1-10	5-5-10	5-12-10	7-21-10
West strut	0.0	0.0	$\frac{1}{8}$ in.	$\frac{1}{8}$ in.	$\frac{1}{8}$ in.
Middle strut	0.0	$\frac{1}{32}$ in.	$\frac{1}{16}$ in.	$\frac{1}{16}$ in.	$\frac{1}{8}$ in.
East strut	0.0	0.0	$\frac{1}{32}$ in.	$\frac{3}{32}$ in.	$\frac{5}{32}$ in.

Transverse Line "B" Crossing Struts.

Date	1-13-10	5-1-10	5-3-10	5-5-10	5-12-10	7-21-10
West strut	0.0	0.0	0.0	$\frac{3}{32}$ in.	$\frac{3}{32}$ in.	$\frac{1}{16}$ in.
Middle strut	0.0	$\frac{1}{16}$ in.	$\frac{1}{16}$ in.	$\frac{1}{32}$ in.	$\frac{1}{32}$ in.	$\frac{3}{16}$ in.
East strut	0.0	0.0	0.0	$\frac{3}{32}$ in.	$\frac{3}{32}$ in.	$\frac{3}{16}$ in.

Transverse Line "C" Crossing Struts.

Date	1-13-10	5-1-10	5-3-10	5-4-10	5-5-10	5-12-10	7-21-10
West strut	0.0	$\frac{1}{4}$ in.	$\frac{1}{4}$ in.	$\frac{5}{16}$ in.	$\frac{11}{32}$ in.	$\frac{13}{32}$ in.	$\frac{1}{2}$ in.
Middle strut	0.0	$\frac{1}{8}$ in.	$\frac{5}{16}$ in.	$\frac{5}{16}$ in.	$\frac{11}{32}$ in.	$\frac{11}{32}$ in.	$\frac{13}{32}$ in.
East strut	0.0	$\frac{3}{16}$ in.	$\frac{1}{4}$ in.	$\frac{5}{16}$ in.	$\frac{11}{32}$ in.	$\frac{11}{32}$ in.	$\frac{5}{16}$ in.

Before beginning work on the large arch, three transit lines at right angles to the axis of the arch were established across the concrete struts just south of Piers Nos. 3, 2, and 1. Table 1 gives the readings taken on these struts, and Table 2 shows the settlement of the piers under their loads. Plate XXIX shows the deformation of half of the large arch, including its centering, and the deformation of one of the small arches and its centering. The record for only one-half of Arch No. 4 is of interest, as the other was quite similar. A record of only one small arch is given, because the settlement of the others was almost identical.

Plate XXIX shows, in tabulated form, the deformation of the falsework under Arch No. 1, the settlement of this arch on the striking of the centering, and the further deformation of the arch after the striking of the centering and during and after the pouring of the columns, spandrels, and floor slabs. It also shows the order of constructing the arch, and the deformation at various times.

It should be noted that the settlement of the falsework under the concentrated loads of the arch-rib blocks was distributed by the sway

FIG. 1.—CENTERING REMOVED FROM ARCHES NOS. 1 AND 2. VIEW FROM POINT NEAR SOUTH ABUTMENT.

FIG. 2.—BACK OF 190-FT. ARCH.

FIG. 1.—CENTERING REMOVED FROM ARCHES NOS. 1 AND 2.

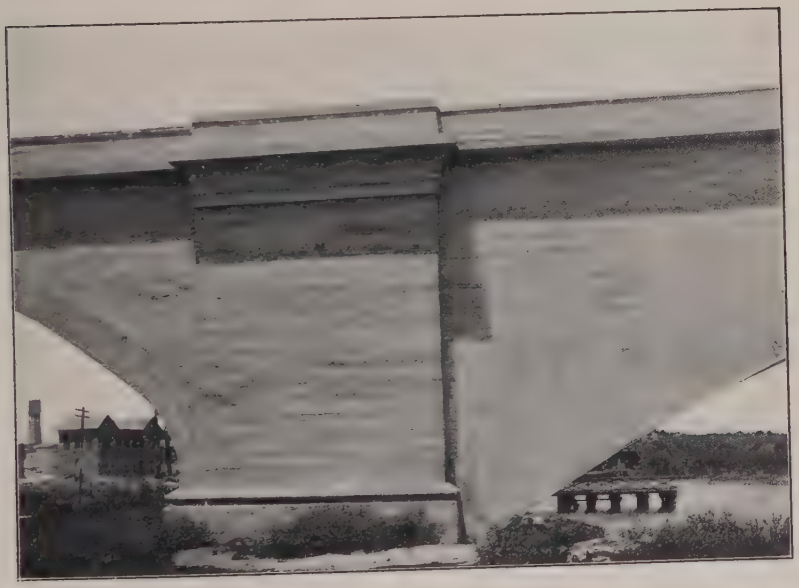


FIG. 2.—PIER NO. 3.

bracing. This is made evident by the fact (line 2 of the tabulation) that, on July 4th, when Points 1, 3, 7, and 9 were only one-half loaded, whereas the other points were fully loaded with the arch-rib load, the settlement under the half-loaded points was almost as great as that under those fully loaded.

After the arch ribs were completed, and before the centers were struck, the additional settlement of the falsework was very small, being between $\frac{1}{8}$ and $\frac{1}{4}$ in. After the centers were struck there was no further settlement of the arch ribs. The centering under Arch No. 1 was struck before the adjacent arch was built, and the survey records show that the striking of the centers did not move Pier No. 1, and this is further demonstrated by the fact that there was no lowering of the crown of Arch No. 1 when the centering was struck.

The total deformation of the falsework was about $1\frac{1}{8}$ in. This allows for approximately $\frac{1}{16}$ in. for take-up of the horizontal joints of the falsework, $\frac{1}{16}$ in. for the bite of the end grain on the side grain, and the remainder was due to the shortening of the piles under their load.

It should be noted that the arch settlements tabulated on Plate XXIX are gross, that is, they include also the pier settlements.

When the other arches were struck there was a tendency of the crown of Arch No. 1 to rise, on account of the thrust of Arch No. 2 against it, and on account of the unbalanced horizontal thrust from Arch No. 4 acting through the three horizontal struts which were designed to transmit this unbalanced thrust to rock.

It will be noted later that Pier No. 1 moved horizontally between $\frac{1}{8}$ and $\frac{1}{4}$ in., due to the thrust of the large Arch No. 4, but the tendency of Arch No. 1 to rise on account of this movement seems to have been counterbalanced to a large degree by the settlement of the adjacent piers and the shortening of the arch ribs under the axial thrust of the added load of spandrels and floor slab.

Plate XXIX also shows the deformation of the falsework of Arch No. 4 and of the arch ribs. The tabulation for Arch No. 4 also shows, more clearly than that for Arch No. 1, the fact that the concentrated loads of the individual arch blocks were distributed through the sway bracing, so that points a considerable distance from those loaded show decided settlement. In other words, it shows that the load was transferred through the bracing into the pile foundations. For instance, note the second line of the tabulation for Arch

TABLE 2.—PIER

Date.	Pier No. 1.				Pier No. 2.				Pier No. 3.			
	23	24	25	26	48	49	50	51	73	74	75	76
7-5-09...	0.009	+0.003	0.003	+0.002	+0.009	+0.009	0.004	0.002	0.004	0.000	0.006	0.013
8-8... ..	+0.003	+0.015	0.006	0.000	+0.013	+0.011	0.014	0.007	+0.003	+0.008	0.002	.....
8-14.....	+0.001	+0.003	0.007	0.004	+0.008	+0.005	0.013	0.011	.....	.....	.....	.....
10-1.....	0.018	0.000	+0.008	+0.006	+0.012	+0.004	0.003	0.001	+0.003	0.002	0.002	0.015
1-21-10..	0.014	.....	.....	0.015	0.012	0.015	0.030	.....	.....	.....	.....	.....
1-22.....	+0.012	0.001	0.014	0.011	0.010	0.010	0.030	0.033	0.006	0.011	0.026	0.037
1-26.....	.....	.....	.....	.....	0.015	.....	0.034	.....	.....	.....	0.022	0.027
1-26.....	.....	.....	.....	.....	.....	.....	0.032	0.033	.....	.....	0.024	0.028
2-21.....	0.013	0.011	0.023	0.021	0.017	0.019	0.036	0.032	0.013	0.014	0.035	0.030
3-7.....	0.007	0.009	0.016	0.011	0.015	0.009	0.031	0.027	0.018	0.033	0.043	0.043
3-18.....	0.007	0.000	0.016	0.008	0.010	0.013	0.029	0.024	.....	.....	.....	.....
4-12.....	0.017	0.016	0.031	0.019	0.022	0.028	0.043	0.039	0.020	0.043	0.046	0.051
4-23.....	0.010	0.014	0.015	0.013	0.025	0.019	0.036	0.029	0.028	0.040	0.035	0.050
5-1.....	0.016	0.021	0.034	0.032	0.028	0.027	0.053	0.045	0.026	0.041	0.047	0.050
5-3.....	.....	.....	.....	.....	.....	0.021	0.041	0.038	0.024	.....	0.044	0.054
5-4.....	0.022	0.029	0.015	0.022	0.023	0.039	0.039	0.036	0.030	0.072	0.053	0.060
5-5.....	0.015	0.021	0.024	0.023	0.029	0.026	0.049	0.037	0.030	0.051	0.053	0.063
5-12.....	0.011	0.020	0.019	0.019	0.030	0.030	0.039	0.036	0.040	0.060	0.055	0.063
5-26.....	0.011	0.020	0.033	0.032	0.032	0.033	0.057	0.055	0.030	0.052	0.061	0.071
7-5.....	.....	.....	.....	.....	.....	.....	.....	.....	0.057	0.080	0.088	0.102
7-20.....	0.021	0.031	0.036	0.035	0.044	0.048	0.075	0.071	0.049	0.080	0.082	0.102
7-21.....	0.022	0.029	0.034	0.036	0.048	0.047	0.075	0.072	0.051	0.077	0.083	0.101

+ Readings indicating a rise of pier are evidently incorrect, as are those showing

No. 4, under date of 2-12-10. This shows that at that time only Points 5, 7, 29, 41, and 53 were loaded, yet nearly all the other points show depression, and, in some cases, as Point 3, it is as great as at the loaded points (Points 5 and 7) in its vicinity. This condition is found throughout the tabulation.

It should be noted that on the date of completing the arch ribs, 3-25-10, the maximum settlement at any point was slightly less than 2 in. and the average about 1¼ in. As the loading of the diaphragms and sheeting was continued, the deformation increased very slightly.

On May 3d, 1910, the centers were struck. There was a considerable lowering of the crown immediately on the striking of the cen-

SETTLEMENTS.

REMARKS.

Arch No. 1.	Arch No. 2.	Arch No. 3.	Arch No. 4.
2 ribs completed, 7-3-09.			
Arch completed, 7-28-09.			
	2 ribs completed, 8-14-09.		
Struck centers, 9-29-09.	Arch completed, 8-17-09.		
		2 ribs completed, 11-8-09.	
	Struck centers. 11-25-09.	Arch completed, 11-10-09.	
			2 ribs completed, 2-20-10.
		Struck centers, 3-8-10.	
Poured spandrels. 4-7-10.			Arch completed, 3-26-10.
	Poured spandrels, 4-19-10.		
Poured floor slab, 4-26-10.		Poured spandrels, 4-30-10.	
			Struck centers, 5-3-10.
	Poured floor slab, 5-16-10.	Poured floor slab, 5-24-10.	
			Poured spandrels. 6-9-10.
			Poured floor slab, 6-19-10.

apparent temporary rises. All settlements are expressed in decimals of a foot.

tering. This amounted to as much as § in., and progressed until the last recorded date, 7-20-10, at which time the bridge was complete, the changes, however, being negligible between 7-5-10 and 7-20-10. The large arch settled more than Arch No. 1, when struck, due to a number of conditions: (1) its greater span; (2) the fact that it was built in a great number of transverse blocks, and that between the adjacent concrete blocks there were hair-thin shrinkage cracks; and (3) that the entire horizontal arch thrust of this large arch was carried through the horizontal concrete struts, causing them to shorten, and the shortening of these struts caused the crown and, in fact, the entire arch rib, to lower.

It will be seen from Table 1 that the shortening of the struts went on progressively also, and, therefore, had material bearing on the lowering of the large arch rib.

In the tabulation for Arch No. 1, and also for Arch No. 4 (Plate XXIX), there are many irregularities in reading, in some cases indicating that the arch went up and down at intervals. It is necessary, therefore, to disregard small differences, which are evidently errors of measurement.

The pier tabulation (Table 2) shows the progressive settlement of all piers. As stated previously, these piers were built on pile foundations, the length of the piles in Pier No. 1 averaging 56 ft. and those in Piers Nos. 2 and 3 averaging 60 and 45 ft., respectively. The general settlement of these piers under their loads was rationally progressive. The four corners of Pier No. 1 settled almost uniformly, whereas Pier No. 2 settled about $\frac{3}{8}$ in. more on the east side than on the west, and Pier No. 3 settled $\frac{5}{8}$ in. more at the northeast corner, Point 76, than at the southwest corner, Point 73. In the case of Pier No. 3, Point 76, the deformation exceeded the theoretic shortening of the piles by about $\frac{1}{2}$ in. It is a matter of conjecture rather than fact whether this settlement was due to any bruising of the piles or to the nosing of their ends into the uneven coral foundation rock.

By noting the dates on which the various ribs and arches were completed, and those when the arches were struck, and other dates contained in the column headed "Remarks," the progressive settlement of the piers will be seen to follow, in a general way, the loads which came on them. Arch No. 1, however, which was struck on 9-29-09, did not seem to affect Pier No. 1 very much, as may be seen by the readings for that pier marked 8-14-09 and 10-1-09. However, after Arch No. 2 was struck (on 11-25-09) as shown by the reading of 2-21-10, the settlement had reached about $\frac{1}{4}$ in.

Pier No. 2 did not show any settlement until after Arch No. 2 was struck. The reading of 1-21-10 shows a settlement of about $\frac{1}{4}$ in. When Arch No. 3 was struck, on 3-8-10, there was no additional settlement (see reading on 3-18-10); but, as the work was built up, the settlement progressed until at completion it was between $\frac{5}{8}$ and $\frac{7}{8}$ in.

Arch No. 4 was begun on 2-8-10 and Pier No. 3 showed considerable settlement when two of its ribs had been completed (as indicated by the reading on 2-21-10). At that time the west side of the pier had settled a little more than $\frac{1}{8}$ in. and the east side about $\frac{3}{8}$ in. When

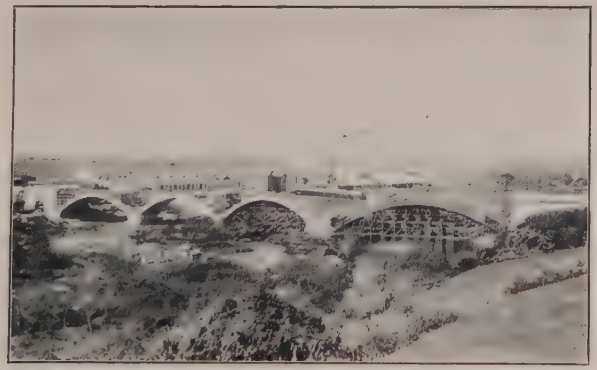


FIG. 1.—ARCHES NOS. 1, 2, AND 3, WITH CENTERING STRUCK.

FIG. 2.—COMPLETED BRIDGE.

FIG. 3.—COMPLETED BRIDGE.

Arch No. 4 was completed the readings on 3-18-10 and 4-12-10 indicate that there was a slight change, showing that this large arch was developing arch action though still on its falsework. After the keying of the arch, there was no further settlement of Pier No. 3 until the center was struck, as shown by the readings from 5-3-10 to the end of the tabulation. It should be noted that the settlement of these piers was gradual, covering a period of a little more than two months, during which time the loads of the columns, spandrels, and floor slab had been added.

Table 1 shows at various times the shortening of the struts under the thrust of the large arch. It should be noted, from Line "C," that the shortening of the struts at the point "C", before striking the centering of Arch No. 4, was about $\frac{3}{16}$ in., and when the striking of the centering was completed, this had increased to $\frac{5}{16}$ in. Two days afterward it was $1\frac{1}{32}$ in., and on the completion of the work it was a little more than $\frac{1}{2}$ in. Assuming the coefficient of elasticity of the concrete at 1 000 000, the total horizontal thrust carried by the struts is approximately 1 800 000 lb. The total horizontal thrust was 5 600 000 lb., leaving 3 800 000 lb. to be taken by the piles, or 7 200 lb. per pile. It is thought that the ground near the piles was able to take such a portion of the total thrust.

It is seen that at Line "B" there was no measurable shortening until after the large arch was struck, in which case the middle strut had shortened $\frac{7}{32}$ in., and the outer struts $\frac{3}{32}$ in. This increased until, on the completion of the work, the middle strut had shortened $\frac{5}{16}$ in. and the outside struts $\frac{3}{16}$ in. A similar shortening is found at Line "A." On the completion of the arches, the shortening of the outside struts was $\frac{1}{4}$ in. and that of the middle strut was $\frac{1}{8}$ in. From Table 1 it appears that Pier No. 1 moved approximately $\frac{1}{8}$ in., Pier No. 2 about $\frac{1}{4}$ in., and Pier No. 3 about $\frac{1}{2}$ in.

As American engineers are doing an increasing amount of concrete work in Cuba and other tropical countries, the unit prices, Table 3, paid to the Contractors, Champion and Pascual, of Havana, are thought to be of interest. The work was let to the lowest bidder, and, as far as the writers have been able to ascertain, these prices were such as to ensure a reasonable profit. They are further of the opinion that the contract cost of similar work executed in the United States, where common labor is based on a 10-hour day and receives, say, $1.50,

and carpenters on form work an average of $3, would be about eight-tenths of the cost in Havana.

TABLE 3.—Unit Prices Paid to the Contractors.

Excavation	Cubic yards...	$1.54	8 950	$13 783.00
Piles	Feet	1.52	36 000	54 720.00
Concrete	Cubic yards...	9.80	12 820	125 636.00
Steel	Pounds	0.035	390 162	13 655.00
Steel pipe, 10 in. in diameter.	Pounds	0.10	6 130	613.00
Drain trap	Each	6.00	44	264.00
Cast-iron piping, 4 and 6 in..	Linear feet....	1.22	1 200	1 464.00
Cast-iron piping, 8 in	Linear feet....	1.83	328	600.24
Cast-iron piping, 10 in	Linear feet....	3.05	52	158.60
Manholes	Each	14.00	10	140.00
Hand-grips	Each	0.50	349	174.50
Tar-paper	Square feet ...	0.07	5 000	350.00
Damp-proofing	Square feet ...	0.047	14 500	681.00
Hand-holes	Each	13.80	18	248.40
Bore-holes, North abutment..	Linear feet....		300	1 593.90
Change in concrete				1 078.13
Extra expansion joints, North approach				32.86
Hinge plates	Square feet....		180	1 558.55
Base mouldings	Linear feet....		576	1 000.00
				$217 751.18

Cost of bridge floor, $6.50 per sq. ft.

In conclusion, the writers wish to call particular attention to the interesting problems involved in carrying a massive structure on generally compressible foundations, and where a large unbalanced horizontal thrust required a special expedient to overcome the insufficiency of the normal pile foundation to resist such a thrust. They wish, however, to make clear that the undesirable condition of caring for the unbalanced horizontal thrust, which might readily have been overcome by arches of equal span, was one which it was not in their power to change.

The bridge was designed by, and constructed under the direction of, the firm of Barclay Parsons and Klapp, of which the writers are members. The pile foundations were placed under the supervision of the firm's representative, Mr. E. F. Saxton. The remainder of the work was built under the supervision of Gratz B. Strickler, M. Am. Soc. C. E., as Resident Engineer, to whom much credit must be given for its satisfactory execution and for the complete records of the arch construction.

The plans for the bridge were made and the work carried on with the co-operation of General Ernesto Asbert, Governor of the Province of Havana, and Mr. Francisco Franquiz, Director of Public Works.

DISCUSSION

L. J. Mensch, M. Am. Soc. C. E. (by letter).—The authors of this excellent paper lay stress on the fact that the bridge was built with a large unbalanced thrust on compressible ground, and that it was of vital importance to design the bridge as light as possible. In the face of the fact that great progress has been made in the application of concrete and reinforced concrete to arch bridges during the last twenty-five years, it is rather surprising that an unusually massive design was used. Mr. Mensch.

It is a fact that properly designed ribbed-arch bridges are in many cases lighter than solid-arch bridges. In the 190-ft. arch, however, the thickness of all ribs and connecting slabs would make a solid arch having an average thickness of more than 3 ft. 6 in. at the crown, and of 5 ft. 3 in. at the abutments.

The adoption of reinforced concrete might have allowed a great lightening of the bridge and of the piers, and also of the unbalanced thrust.

In an arch bridge of 200 ft. span and 35 ft. rise, described by the writer,* the ribs are 8 ft. $10\frac{3}{4}$ in. from center to center, and the thrust is about 625 000 lb., which is one-half of the thrust shown in Fig. 1.

The adoption of struts was a good expedient, which, like that of ties, is only too rarely used. There must have been very weighty reasons for the omission of sheet-piling around the piers, and, as it was omitted, the writer thinks the ground must have been very much better than would appear from the description in the paper.

The authors' remarks in regard to the design of the arches by the common or static theory and by the elastic theory are likely to disappoint those who wish to be instructed by such a paper on a subject about which so much mystery and confusion seem to exist. While it takes some time and effort to master the elastic theory, probably too much for the busy engineer to acquire if he did not learn it at school, its results for the design of arches are so simple that their application is within the reach of every engineer.

The writer will endeavor to give these results in as simple a way as possible, and hopes that it will help to clear a subject which is generally so little understood.

(1) The elastic theory teaches that the most favorable line of pressure is obtained when the dead load and one-half of the live load, uniformly distributed over the entire arch, is used in finding the funicular polygon. While the line of pressure, in a bridge with open spandrel construction, like the one in question, is very closely a parabola, it deviates considerably in the case of earth-filled arches. The

* "Reinforced Concrete Pocketbook," p. 177.

Mr. Mensch.

writer has succeeded in evolving a very simple method of finding the line of pressure for earth-filled arches, and the ordinates of the pressure lines can be taken from Table 4, in which a denotes the weight of the bridge per square foot at the crown plus one-half of the live load, which is usually taken as one-half of 100 lb.; f denotes the weight of the bridge per square foot at the springing line; and H is the horizontal component of the thrust.

TABLE 4.

(1) $\frac{a}{f}$	(2) $\frac{H}{l^2}$	Ordinates at: (3) 0.4 l.	(4) 0.3 l.	(5) 0.2 l.	(6) 0.1 l.	(7) Tan. b.
0.50	8.33	0.969 f	0.878 f	0.697 f	0.412 f	5.00 $\frac{f}{l}$
0.45	7.71	0.970 "	0.879 "	0.703 "	0.418 "	5.08 "
0.40	7.09	0.971 "	0.880 "	0.706 "	0.426 "	5.18 "
0.35	6.46	0.972 "	0.885 "	0.715 "	0.435 "	5.32 "
0.30	5.83	0.975 "	0.887 "	0.722 "	0.442 "	5.41 "
0.25	5.21	0.976 "	0.889 "	0.733 "	0.453 "	5.60 "
0.20	4.58	0.978 "	0.901 "	0.745 "	0.464 "	5.81 "
0.15	3.96	0.985 "	0.913 "	0.763 "	0.482 "	6.11 "
0.10	3.33	0.987 "	0.924 "	0.784 "	0.505 "	6.59 "
For Parabola.		0.96 "	0.84 "	0.64 "	0.36 "	4.00 "

The figures in Column 2, multiplied by the square of the span, in feet (and without any drawing whatsoever), gives the thrust, in pounds per linear foot of bridge. This line of pressure has the property that the maximum positive bending moments at the loaded side have the same values as the maximum negative moments at the unloaded side.

(2) Where the bending moment from concentrated loads is smaller than that caused by a moving uniformly-distributed load, it is common practice to calculate the arch for one-half the span uniformly loaded. The new line of pressure is lowered at the springing line of the loaded side a maximum distance, m, approaches the old line of pressure, and intersects it at a point one-eighth of the span distant from the abutment, remains above this line for the remaining portion of the half span, and intersects it at the crown, remaining below it until seven-eighths of the span is reached, and rises a maximum value, m, at the abutment of the unloaded side. The exact vertical deviations of the new line of pressure can be obtained very simply, as shown in Fig. 4. Over each half span is drawn a parabola, the rise of which is $m = \frac{wl^2}{64\ H}$ in feet; in which H is the horizontal component of the thrust, in pounds, from dead load and half of the live load uniformly distributed; l is the span, in feet; and w is the uniform live load per square foot,

or per linear foot in case of ribbed arches. $A\ D$ and $B\ E$ are made equal to $\frac{wl^2}{64\ H}$, and then the line, $D\ E$, is drawn. The verticals of the shaded areas are the vertical deviations of the pressure line. Those which lie above $D\ E$ have to be taken as the rise of the new pressure line, and those below that line indicate a lowering of the pressure line. It is easily found that the maximum deviation, except at the springing line, occurs at a point five-sixteenths of the span distant from the abutment, and is equal to $\frac{9}{16} \times \frac{pl^2}{64\ H}$; or, the greatest bending moment is $\frac{9\ pl^2}{1\ 024}$, in foot-pounds.

As the stresses can be found from the bending moment, it is not necessary to draw the diagram, Fig. 4, or to show the change of the line of pressure in the elevation of the arch. Most of the concrete and stone bridges which have been built during the last twenty-five years were designed by this method.

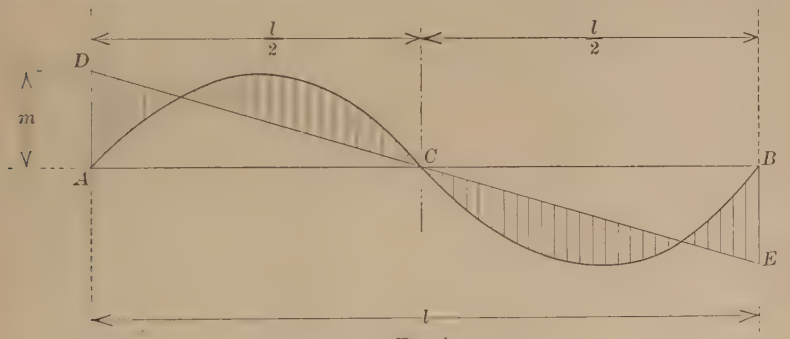


FIG. 4.

(3) The change of the line of pressure produced by a group of concentrated loads cannot be found by any other method than that of the elastic theory. Methods* applicable to this case do not offer more difficulty than those used for simple beams. The busy engineer, however, will welcome a table of equivalent uniform loads to be applied on the half-span of the arch, which will produce the same maximum deviation of the line of pressure as a group of concentrated loads. For a 10-ton wagon, according to Cooper's Specifications, the equivalent loads per square foot may be found from Table 5.

TABLE 5.

Span, in feet	30	40	50	60	70	80	90	100	110
Load, per square foot	265	205	170	160	150	137	125	114	109
Span, in feet	120	130	140	150	160	170	180	190	200
Load, per square foot	104	97	91	85	82	78	74	71	68

* In the "Reinforced Concrete Pocketbook."

Mr. Mensch.

For heavier wagons, the uniform loads may be increased in proportion.

(4) The effect of the shortening of the arch is very small in properly designed reinforced concrete arch bridges, and, as a rule, can be neglected, except where the rise of the arch is one-eighth of the span or less, and especially where ribbed arches are used. The effect of the shortening of the arch on the line of pressure when one-half of the span is loaded, is to elevate it at the crown a distance, $c = \frac{1}{3} \times \frac{d^2}{f}$, in feet, and to lower it at the springing line of the loaded side a distance, $c_1 = \frac{11}{12} \times \frac{d^2}{f}$, where d equals the depth of the arch or arch ring at the crown, in feet, and f equals the rise of the arch, in feet.

For example, assume a 140-ft. arch of 25 ft. rise, and having a thickness of 2 ft. at the crown; $c = \frac{2^2}{3 \times 25} = 0.0533$ ft., and $c_1 = \frac{11}{12} \times \frac{4}{25} = 0.1467$ ft., which are very small values.

When both halves of the arch are uniformly loaded, c remains $= \frac{1}{3} \times \frac{d^2}{f}$, while $c_1 = 2\,c$.

Inasmuch as the dead load, as well as the live load, will cause a shortening of the arch, it follows that it is, strictly speaking, impossible to design an arch in which the line of pressure coincides with the center of gravity of the sections, and that there is always a deviation from the ideal line of pressure.

In the case of the 190-ft. arch of the Almendares River Bridge, $c = \frac{5.5^2}{3 \times 32.5} = 0.31$ ft., and $c_1 = \frac{11}{12} \times \frac{5.5^2}{32.5} = 0.85$ ft., which is about twice the deviation of the line pressure caused by the live load alone.

The bending moment due to the shortening of the arch equals the horizontal component of the thrust multiplied by c; or, taking the value of the thrust from Fig. 4, $1\,250\,000 \times 0.31 = 387\,500$ ft-lb. at the crown, and $1\,250\,000 \times 0.85 = 1\,060\,000$ ft-lb. at the springing line. While the first value is about as great as the bending moment produced at five-sixteenths of the span from the live load, the value at the abutment is considerably greater than the bending moment produced by the live load direct. From the paper it does not appear that the authors took the effect of the shortening of the arch into consideration, as in this case the line of pressure will deviate at the crown from that obtained by the static theory.

The stresses produced by the moment at the abutment

$$-\frac{1\,060\,000 \times 12}{54 \times \frac{102^2}{6}}, \text{ or } 136 \text{ lb. per sq. in.}$$

Mr. Mensch.

In the case of a solid arch having a thickness of 3 ft. 6 in. and 5 ft. 3 in. at the crown and abutment respectively, $c = 0.0796$ ft. and $c_1 = 0.29$ ft., or a stress of 122 lb. per sq. in. at the springing line.

(5) The effect of a change of temperature is very similar to the effect of the shortening of the arch. Let t = the change of temperature, in degrees Fahrenheit; e = the coefficient of expansion for 1° Fahr., generally taken as 0.0000050; I = the moment of inertia of the crown section; f = the rise, in inches; and E the modulus of elasticity. Then the change of the thrust, $\Delta H = \frac{45\,E\,I\,e\,t}{4\,f^2}$.

If this thrust is supposed to act in a horizontal line at a height of $\frac{2}{3}\,f$ above the intersection of the line of pressure with the abutment, as shown in Fig. 5, the leverages of the moments produced by the change of thrust can be scaled off for every point of the arch. The latest practice assigns to E a value of 1 500 000, and to t a value of 40° Fahr.; hence

$$\Delta H = \frac{45}{4} \times 1\,500\,000 \times 0.000005 \times 40\,\frac{I}{f^2} = 3\,375\,\frac{I}{f^2}\text{ lb.},$$

which means an increase of thrust for a rise of temperature, and a decrease of thrust for a fall.

The greatest moment from a change of temperature occurs at the springing line, and is $2\,250\,\frac{I}{f}$ in-lb.

At the three-tenths point of the arch, where the greatest stresses from the live loads usually occur, the bending moment is $584\,\frac{I}{f}$ in-lb.; at the crown it is $1\,125\,\frac{I}{f}$.

For example, take the 140-ft. arch, of 25 ft. rise, and 24 in. thick at the crown. $I = \frac{12 \times 24^3}{12} = 13\,824$ in-lb., $f = 300$ in., and $\Delta H = 3\,375\,\frac{13\,824}{300 \times 300} = 518$ lb., which produces a uniform pressure of less than 2 lb. per sq. in. at the crown.

The moment at the crown $= 518 \times \frac{25}{3} = 4\,320$ ft-lb., which would influence the design of the arch if the greatest bending moment from the live load would occur at the crown. At the three-tenths point of the arch the bending moment from a change of temperature $= 584 \times \frac{13\,824}{300} = 26\,800$ in-lb. The maximum stresses at this section, assuming the thickness of the arch to be 27 in., $= \frac{26\,800}{12 \times \frac{27^2}{6}} = 18.4$ lb. per sq. in.

Mr. Mensch.

At the springing line the moment $= 518 \times \frac{2 \times 25}{3} = 8\,640$ ft-lb., and maximum stresses $= \frac{8\,640 \times 12}{12 \times \frac{36^2}{6}} = 40$ lb. per sq. in.

For the 190-ft. ribbed arch, I is approximately 1 290 000 in-lb., $f = 390$ in., and $\Delta H = 3\,375 \times \frac{1\,290\,000}{390 \times 390} = 28\,700$ lb., or about 8 lb. per sq. in. The moment at the crown $= 28\,700 \times \frac{1}{3} \times 32.5 =$ 311 000 ft-lb. The moment at the abutment $= 28\,700 \times \frac{2}{3} \times 32.5 =$ 622 000 ft-lb. The moment at the three-tenths point of the arch $= 548 \times \frac{1\,290\,000}{390} = 1\,935\,000$ in-lb., corresponding to a stress of only 38 lb. per sq. in. Similarly, the maximum stresses at the springing line are 78 lb. per sq. in.

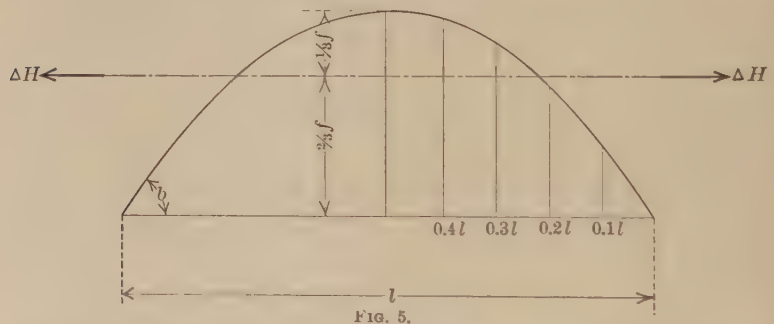


FIG. 5.

For a 190-ft. solid arch, $\Delta H = 14\,700$ lb., and the maximum stresses at the springing line are 52.8 lb. per sq. in.

As pointed out by George S. Webster and Henry H. Quimby, Members, Am. Soc. C. E., in their valuable paper on the Walnut Lane Bridge in Philadelphia,* the actual change of temperature of the concrete and the accompanying rise and fall of the arch depend on the mean weekly temperature, or probably the mean monthly temperature; and, while the highest and lowest temperatures of the air during their observations varied between 95 and 10°, the corresponding temperatures of the mass of the arch were 73 and 31°, a change of only 42° Fahr. When the concrete of the arch is completed at a time when the mean weekly temperature is about the mean of these values, a change of temperature of only about 20° may be assumed, and the stresses above given are reduced to one-half.

* *Transactions*, Am. Soc. C. E., Vol. LXV, p. 423.

Mr. Mensch.

The change of elevation at the crown due to a change of temperature of 40° can be found according to the elastic theory if the moments shown in Fig. 5 are used in the formula for deflection, and is equal to $\frac{1}{11\,700} \times \frac{l}{f} \times L$, and this theoretical value, contrary to the statement of the authors, agrees very well with the observations from quite a large number of important bridges built in voussoirs, both in America and abroad.

(7) The deflection of the crown of the arch due to the dead load and uniformly distributed live load, $\Delta f = \frac{17}{64} \times \frac{l}{f} \times L \times \frac{s}{E}$, in feet, where L is the developed length of the arch, in feet, and s is the compressive stress of the crown section.

For $s = 250$ lb. per sq. in., $\Delta f = \frac{1}{22\,600} \times \frac{l}{f} \times L$.

The deflection from a uniformly distributed load over the whole arch can be found by the same formula, if for s is substituted the additional stress caused by the load; and, when only one-half of the span is loaded, the deflection of the crown is one-half of the value obtained for the full load.

For a point distant five-sixteenths of the span from the abutment, the deflection produced by a uniform load on the half span $= \frac{1}{24\,000} \times \frac{pl^4}{EI}$.

(8) The effect of shrinkage on the line of pressure is a subject on which very little can be found in the literature on concrete arches. Very careful tests made on small and medium-sized concrete specimens show that the effect of shrinkage after 1 month is the same as a fall of temperature of 60° Fahr., and that, after from 3 to 6 months, the effect of the shrinkage is the same as a fall of 100° Fahr., which would mean that the stresses in the arch at the time when the centering is struck are about one and one-half times the stresses given under Section (5) for a change of temperature of 40°, and that these stresses increase to two and one-half times those values after from 3 to 6 months. While these stresses may exist in bridges having a thickness of only a few inches, they do not exist in arches even only 12 in. thick, which has been found by the writer by the following observation. In a number of bridges the centering did not settle after the arch ring was completed, nor was the slightest movement of the abutment observed. After the centering had been struck and the deflection of the crown had been carefully observed for a period of a month or two, the deflection was computed for dead loads, according to Section (7), and for a change of temperature, as given in Section (5). The remaining deflection must have been due to the shrinkage of the concrete,

Mr. Mensch.

and, in one instance, was found by the writer to amount to only six-tenths of that obtained for a change of 40° Fahr.

The fact that the shrinkage of large masses of concrete is considerably less than shown in small test specimens is borne out by the experience of the authors when they state that the struts, having a length of about 306 ft., showed a shrinkage of only $\frac{3}{16}$ in., or only about $\frac{1}{19\,500}$ of the length.

(9) The effect of spreading of the abutments on the line of pressure is identical with the effect of a fall of temperature. A fall of temperature of 40° corresponds to a spreading of the abutments of $\frac{1}{5\,000}$ of the span. In the 190-ft. span, the spreading of Pier No. 3 amounted to $\frac{9}{16}$ in., or $\frac{1}{4\,000}$ of the span, corresponding to a stress 25% greater than for a fall of 40° Fahr., or to 98 lb. per sq. in. at the springing line. For a solid arch having a thickness of 5 ft. 3 in. at the abutment, the maximum stresses caused by the spreading of the foundation at the springing line would amount to 66 lb. per sq. in.

(10) The relative lowering, Δy, of one abutment in reference to the other, causes at the springing line a stress equal to $\frac{E d_1 \Delta y}{l^2}$ in pounds per square inch, where d_1 is the depth of the arch at the abutment, in inches.

In the 190-ft. arch, Δy was $\frac{9}{16}$ in., $d_1 = 102$ in., or the maximum stresses $= 1\,500\,000 \times \frac{102}{190 \times 12 \times 190 \times 12} \times \frac{9}{10} = 26.8$ lb. per sq. in.

In the case of a solid arch, the stresses would amount to only 16.6 lb. per sq. in.

In any arched bridge it is clearly of the utmost importance to make careful investigations to determine the horizontal and vertical movements of the abutment at the springing line before the final design of the bridge. These investigations should include tests of piles and test loads on the ground; and, from settlement under the future unit loads, a foreknowledge of the movement of the abutment can be obtained.

(11) From the foregoing it clearly follows that, in bridges having a comparatively low rise, or in arches having deep ribs, many of the stresses are directly proportional to the moment of inertia of the crown section; and, while ribbed arches may appear to be more favorable for the live-load stresses, the stresses from the shortening of the arch, from a change of temperature, from the shrinkage of the concrete, and from a settlement and spreading of the abutments, are far greater than in a solid arch, and bridges with insecure foundations,

like the one described in the paper, should certainly have been built with solid arches; or, preferably, a reinforced concrete arch should have been used. Mr. Mensch.

The adoption of a three-hinged arch was quite unnecessary in this case, as the greatest stresses at the springing line of an arch without hinges would have amounted to only 669 lb. per sq. in., namely:

Direct compression from the thrust	260 lb.	(260)
Positive or negative stress from the live load	70 "	(92)
Positive or negative stress from the shortening of the arch	136 "	(122)
Positive or negative stress from a change of temperature (a change of temperature of only 20° was assumed)	39 "	(26)
Negative stress from the shrinkage of the concrete (assumed to be 50% of the effect of the change of temperature of 40°)	39 "	(26)
Positive or negative stress from a spreading of Pier No. 3	98 "	(66)
Positive or negative stress from the settlement of the pier	27 "	(17)

Or a maximum positive stress of 669 lb. per sq. in.
And a maximum negative stress of 155 lb. per sq. in.

This could easily have been taken care of by a small enlargement of the ribs at the springing line, or by a small amount of steel reinforcement. It is thereby clearly shown that the line of pressure was outside of the middle third, and that the stresses which are ordinarily neglected, but which can be calculated in such a simple way, are of vital importance in the design of concrete arches, and while a small spreading of the abutments might easily increase the negative stresses in a concrete arch to a point where the concrete cracks, a reinforced concrete arch will allow a much greater abuse before unsightly cracks can appear.

In a solid arch having a thickness of 5 ft. 3 in. at the springing line, the greatest positive stress would have been 609 lb. per sq. in. and the greatest negative stress would have been only 89 lb. per sq. in.

The writer questions whether the hinges in the 190-ft. arch really acted as expected. All the influences above mentioned cause a considerable change of angle, both at the crown and at the springing line, and these evidently must have produced a considerable leverage of the thrust at the contact surfaces. The authors state that the changes in the hinged opening of the large arch were about $\frac{1}{16}$ in. It seems to the writer that this change of a total of about $\frac{1}{8}$ in. might have been distributed over the whole span in an arch without hinges with only a small additional stress.

Mr. Mensch.

A change of $\frac{1}{16}$ in. at the edge of the concrete section corresponds to a change of $\frac{1}{40}$ in. at the edge of the 2-in. cast-iron plate, or to a stress of 60 000 lb. per sq. in., which the concrete in contact with the edge evidently could not carry, and there must have been a local pulverizing of the concrete.

Mr. Strickler.

G. B. STRICKLER, M. AM. SOC. C. E. (by letter).—As stated by the authors, the construction of the Almendares River Bridge was begun in November, 1908, and, prior to the middle of June, 1909, the pile-driving for the piers and struts was done, the concrete struts built, and the concrete for the south abutment and Piers Nos. 1, 2, and 3, carried up to Elevation 22.87. At this point the writer, as Resident Engineer for Messrs. Barclay Parsons and Klapp, took charge of the work.

The forming for the ribs, sheathing, and diaphragms for Arch No. 1 was then about completed, and the concrete work on this arch was begun on June 30th. The supporting framework for the centering was well done and the lagging properly laid, but the side forms for the ribs were set up and wired and nailed in place, and, on pouring the concrete, these side forms could not be kept in line, and the bulging noted by the authors resulted. The wiring was eliminated for similar work in Arch No. 2, but too much dependence was placed on nailing, rather than bolting, and the results were again unsatisfactory. These details were improved in the case of the other arches, and for the ribs of Arch No. 4 and the exterior walls built thereafter, bolts were used, passed through cement sleeves, which admitted of their removal later. This was accepted in lieu of the specification requirement which provided for sleeve nuts with ends placed not closer than 3 in. to exposed surfaces, the holes being pointed up with mortar after the removal of the bolts.

The cement sleeves were fairly satisfactory, but, owing to the variation in color between their exposed ends and the surrounding concrete, together with the difficulty in moulding the ends of the sleeves truly normal to their length, and later placing them in the same plane with the exposed surfaces, the sleeve nuts as specified are much to be preferred.

On this work, as is often the case, it was difficult to secure a thorough study and detailing of the form work prior to its construction. It has been the writer's experience that such conditions lead to added expense as well as unsatisfactory work, and in writing specifications for concrete work, it would probably be difficult to lay too much stress on this feature. In the average case the services of an experienced and capable draftsman would not amount to more than 2% of the pay-roll expense, and to say that 10% could thus be saved is believed to be conservative. When this is supplemented by

Mr. Strickler.

a well-designed construction plant, much has been done toward reducing the cost of modern concrete work.

The concrete, as well as the form work, was improved as the work progressed. The authors have referred to the stone, which, owing to the numerous clay-filled cavities, was difficult to wash and prepare for use. These water-worn cavities, prior to crushing, were from 1 in. to 1 ft. or more in diameter, and those operating the crusher plant had difficulty in securing a proper selection and cleaning of the stone before crushing. When the stone was crushed, most of the clay, if dry, went into the finer sizes, and, therefore, these were necessarily excluded from the work. As a matter of interest, some of the screenings were washed and found to contain 30% of clay.

The concrete in Arch No. 1 probably contains from 12 to 13% of clay. It is believed that the clay in the concrete of Arch No. 2 does not amount to more than from 7 to 8%, with some further improvement in the later work. At the beginning of the arch construction a barrel filled with water was kept near the super-hopper containing the stone just prior to its being drawn into the mixer, and before being used several bucketsful were thrown over the charge and allowed to drain off. This was unsatisfactory, and two perforated pipes were laid on the sloping bottom of the hopper. These, as well as a jet pipe, were connected with a steam pump which gave a pressure at the outlets of about 20 lb. per sq. in. The valve controlling the perforated pipes was opened when beginning to fill the hopper, and, after filling, the jet pipe was used and the washing continued until the water draining off was reasonably clear.

When the forms were removed from the ribs of Arch No. 1, it was found that in three places, where leaks had occurred, voids in the stone were unfilled. On careful investigation these planes (almost truly horizontal) were found to extend into the work 2 ft. or more. These sections were cut out and replaced, and this operation showed clearly the effects of the clay. Such planes of weakness near the springing of an arch would be particularly hazardous. The concrete in Arch No. 1 was studied closely during the ensuing year, and was found to harden less rapidly than that containing less clay.

In placing concrete in work of this character, where the concrete is made quite wet and there is an interval of 1 min. or more between batches, great care should be taken, after depositing each batch, to spade or otherwise work it deep enough to make sure that it will be well mixed with the batch below. This will also tend to bring the light inert parts to the top, where they will remain or run off with the excess of water; and, in cases where the top of the form is not reached within a day's work, they may be cleaned off before resuming work.

Mr. Strickler.

The 2-in. pipes through the lower edges of the diaphragms, just above the soffit sheathing, were found to be entirely too small; not that they failed to pass the water when they were open, but, being so small, they were easily clogged. A diameter of 2 in. would be ample for the openings through the sheathing adjacent to the piers and abutments, as they could be opened readily from the ground in cases of stoppage, but, through the diaphragms, such openings should be not less than 8 in. in diameter, and larger ones would be even better. It is also believed that, for constructional reasons, diaphragms deeper than 3 ft. should be not less than 12 in. thick.

In building columns, the writer has found that the best results are obtained by not filling them too rapidly. By putting in 2 ft. at a time, with about 15 min. between successive fillings, time enough is provided for rodding it well and stopping any leaks which may have developed. The leaks must be looked after closely, and, after stopping, in each case, the concrete should be again rodded to insure the refilling of any voids.

In building ribbed arches such as those under discussion, the writer is strongly of the opinion that no arch action should be permitted in any of the ribs of a given arch until all are built and made ready for the keying. This was particularly desired in the 190-ft. span of the Almendares Bridge, but the work was done during the season when freshets are to be expected, and the contractor was afraid of the risk. The author's reference to the lipping of the sheathing concrete adjacent to the older ribs, clearly shows that, to some extent, they were off the centering. Ribs built and keyed as this arch was, from the longitudinal center outward, are undoubtedly favorable to the centering, increasing as the arch action in the older ribs is developed; but, in addition to the possible harm done the concrete, is not the arch to some extent "center bound"?

Referring to the tell-tales or observation points used in connection with the rib construction of all arches, it should be noted that these were all established over the respective arches prior to beginning the concrete work, and were carried by the lagging. The lagging was particularly well laid over properly curved joists, but when the concrete was laid, say, from the longitudinal center outward, it was found that the lagging raised somewhat at the ends after being weighted at the center, which accounts for most of the plus readings obtained.

The centers were all struck from haunch to crown, first lowering from $\frac{1}{4}$ to $\frac{3}{8}$ in., symmetrically, that is, in similar bents at the same time, until the crown was reached, and this was repeated until well clear of the concrete. Attention is called to an inadvertent statement to the effect that the centering was struck before laying the concrete in an adjacent arch. The order in which this was done may be

noted by reference to the remarks on Table 2, where it will be seen that Arch No. 2 was completed on 8-17-09, and the centering under Arch No. 1 was struck on 9-29-09. Arch No. 3 was completed on 11-10-09, and the center under Arch No. 2 was struck on 11-25-09. The two central ribs of Arch No. 4 were completed and keyed on 2-20-10, the next two on 3-6-10, and the center under Arch No. 3 was struck on 3-8-10. Mr. Strickler.

The plans and details for this work were well and carefully made, and the specification was practical, fair, and brief, but well to the point.

William Barclay Parsons, M. Am. Soc. C. E.—Although it may be improper for the speaker to discuss the merits of the paper, it is entirely proper for him to refer to one detail, the credit for which is entirely due to Mr. Klapp, and that is the very ingenious solution of the problem of taking care of the unbalanced thrust between the large arch and the neighboring small arch. As the authors have stated, the first investigations led the engineers to believe that very good material on which to found the piers would be found within the reach of short piles. Mr. Parsons.

When a fuller study was made the rock was found only at a depth of 60 or 70 ft. or more from the surface, and the overlying ground was of such poor quality that little dependence could be placed on it; and, further, the rock surface, as the paper shows, was inclined downward on the north side, in the direction which would be taken by batter piles.

Those who have worked at all in that coral rock know what it is. Its surface is sometimes smooth, at other times very porous, and always quite irregular, so that, if a batter pile is driven apparently to the limit, there is the probability that, when a heavy load is put upon it, the rock upon which it is supported will give way, or the batter pile itself will slip.

There was one time when so much difficulty was encountered in obtaining a satisfactory foundation within the permissible limit of cost that the bridge might have been given up. It was then that Mr. Klapp suggested resolving the unbalanced thrust into two components, taking care of one of them by vertical piles, and then carrying the other back to the rock on the south side. That was done, and the struts described in the paper were put in place.

Had batter piles been used, 425 would have been required in this one pier, whereas only 256 were needed, and the quantity of concrete was also reduced. The cost of the work was very much lessened, and a foundation was produced that was absolutely reliable, because, as described in the paper, from measurements taken during construction, those struts took up the pressure, as they were expected to.

Mr. Parsons. It may be also proper to point out that, although the cost of the bridge was somewhat more than that of one of similar character in New York, yet it was a cheaper bridge than if built of steel, when the Cuban duties were added to the cost of the steel; and the maintenance charge will be very much less than it would have been with a steel structure.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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TRANSACTIONS

Paper No. 1211

SOME OF THE PROPERTIES OF OIL-MIXED PORTLAND CEMENT MORTAR AND CONCRETE.*

By Logan Waller Page, M. Am. Soc. C. E.

With Discussion by Messrs. S. D. Newton, T. S. Oxholm, Lars R. Jorgensen, H. de B. Parsons, Frank J. Phelan, Clifford Richardson, R. D. Coombs, Maurice G. Parsons, E. H. Thomes, H. F. Dunham, J. B. Lippincott, E. Kuichling, and Logan Waller Page.

In October, 1909, the writer found that oil in considerable quantities could be combined with wet Portland cement paste by a simple mixing process. It was noted that when oil was added to wet cement it disappeared completely when mixed for a few moments, and, furthermore, that there was no apparent separation of the ingredients upon setting. Experiments were at once begun with various kinds of oil, to ascertain which was best adapted for the mixture. After these preliminary enquiries were completed, the following tests were conducted, or are now in progress, in order to investigate the properties of oil-mixed cement mortar, and concrete:

(1) The tensile strength of neat cement and 1:3 mortar;
(2) The time of setting;
(3) The crushing strength of mortar and concrete;
(4) Impact tests on concrete;
(5) The modulus of elasticity of concrete;
(6) The absorption of concrete;
(7) The permeability of concrete;
(8) Bond tests;
(9) The density of concrete.

* Presented at the meeting of November 1st, 1911.

Some of these tests are in an incomplete state; others are contemplated, in an effort to develop those phases of the material which give most promise of practical results. The writer's excuse for publishing them in such an incomplete state is that, with his limited facilities for testing, it would require considerable time to complete the work, and, in view of the results already obtained, it seemed likely that engineers with better facilities would hasten the work through its experimental stage.

Materials.—The materials used consisted of Portland cement, river sand, and ¾-in., crusher-run gneiss. Analyses of the several oils selected for use are given in Table 1. The cement complied with the specifications recommended by the Special Committee on Concrete and Reinforced Concrete. Mechanical analyses, typical of the sand and stone used, are given in Table 2.

The sand contained 37% of voids, the stone 43 per cent.

Method of Mixing Concrete.—The weights per cubic foot of the dry sand and stone were determined, the materials being loosely shaken in a measure containing 1 cu. ft. The weight of cement was assumed to be 95 lb. per cu. ft. The percentage of moisture in the sand and stone was determined every day that a concrete mixture was made, and the proportions of sand and stone in the mixture were corrected

TABLE 1.—Analysis of Oils Used

Sample number	4 145	4 146	4 147
Type	Fluid Residual Oil.	Fluid Residual Oil.	Fluid Residual Oil.
Character	Fluid, greasy.	Fluid, greasy.	Fluid, greasy.
Specific gravity at 25°/25° cent	0.924	0.910	0.926
Loss at 163° cent., 5 hours, 20 grammes.	6.86%	12.56%	7.98%
Character of residue	Fluid, greasy.	Fluid, greasy.	Fluid, greasy.
Soluble in CS_2, air temperature	99.99%	99.99%	99.93%
Organic matter insoluble	0.01	0.01	0.07
Inorganic matter insoluble	0.00	0.00	0.00
	100.00	100.00	100.00
Percentage of total bitumen insoluble in 86° paraffin naphtha	2.23%	6.82%	10.16%
Fixed carbon	2.41	3.36	5.11
Specific viscosity, Engler 50° cent	14.2	6.4	18.2

for the moisture content. All concrete was proportioned by volume, but the materials were measured by weight. In all cases the percentage of oil used is based on the weight of the cement in the mixture.

The following method of mixing was used: The proper quantities of cement and sand were mixed dry until of uniform color. Water was then added and the mass was turned as often as was necessary to produce a rather wet mortar. The water added to the mortar was not weighed, the appearance of the mix being the criterion for determining the proper quantity. In general, however, the total quantity of water, including that in the sand, was about 17% of the total weight of the cement and sand. After having mixed the mortar, the desired quantity of oil was added, and the mass was turned over as many times as was necessary to break up the oil and incorporate it thoroughly with the other ingredients. The previously moistened stone and the oil-cement mortar were then combined and turned until thoroughly mixed. The percentage of water required to make a rather soft mixture was determined from the appearance of the first batch, and every batch of the same proportions made thereafter contained the same gross quantity of water.

All mortar compression test pieces were proportioned by weight, the sand being thoroughly dry; 19.5% of water was used in the mix.

IN CEMENT-CONCRETE MIXTURES.

4 149	4 170	4 923	4 981	5 053
Fluid Residual Oil.	Cut-back Oil Asphalt.	Fluid Residual Oil.	Fluid Residual Oil.	Fluid Residual Oil.
Fluid, greasy.	Fluid, sticky.	Fluid, greasy.	Fluid, slightly sticky.	Fluid, slightly sticky.
0.923	.....	0.945	0.898	0.924
7.02%	18.38%	1.35%	27.17%	3.70%
Fluid, greasy.	Semi-solid, sticky.	Fluid, greasy.	Slightly granular; more viscous than original.	Fluid; granular in appearance.
99.95%	99.81%	99.96%	99.95%	99.90%
0.05	0.13	0.04	0.02	0.07
0.00	0.06	0.00	0.03	0.03
100.00	100.00	100.00	100.00	100.00
2.24%	16.87%	3.46%	1.00%	4.12%
1.98	.....	4.18	1.77	2.82
20.2	.....	65.1	2.5	17.4

TABLE 2.—Mechanical Analyses of Sand and Stone.

Sand.		Stone.	
Sieve.	Percentage retained.	Sieve.	Percentage retained.
¼-in.	3	¾-in.	28.4
⅛-in.	11	½-in.	66.3
No. 10	17	¼-in.	92.1
No. 20	42	⅛-in.	98.4
No. 30	66		
No. 40	87		
No. 50	93		
No. 80	96		
No. 100	99		

1:3 Mortar.—Tests made on 1:3 mortar containing oil show that the tensile strength is influenced very little by the oils at 7 days, 28 days, or 6 months, and furthermore, there is a decided gain in strength at 28 days and 6 months over that at 7 days. The strength of oil-mixed mortars is given in Table 3, and plotted on Fig. 1.

TABLE 3.—Tensile Strength of 1:3 Mortar, Ottawa Sand.

Percentage of oil.	Oil No. 4 923.			Oil No. 5 053.		Oil No. 4 981.	
	7 days.	28 days.	6 mos.	7 days.	28 days.	7 days.	28 days.
0	256	296	326	244	331	283	367
2½	299	400	449	297	353	259	327
5	287	316	372	313	334	268	341
10	252	331	360	304	371	264	329

Time of Setting.—The addition of oil retards both the initial and final set. Quantitative results of tests with the Gillmore needles on specimens subjected to identical conditions are given in Table 4 and are shown graphically on Fig. 2. The effect of 10% of oil is to increase the time of initial set about 90%, and the time of final set about .60 per cent.

TABLE 4.—The Effect of Oil on the Time of Setting of Portland Cement.

Oil No. 4 923.

Percentage of oil	0	2½	5	10
Initial set	1 h. 18 m.	1 h. 31 m.	1 h. 57 m.	2 h. 27 m.
Final set	3 h. 43 m.	4 h. 56 m.	5 h. 27 m.	5 h. 57 m.

Crushing Strength.—Specimens of mortar and concrete were moulded in cylindrical forms, 6 in. in diameter and 6 in. high. The materials were deposited in the moulds in layers about 2 in. thick, and were troweled and tamped. The moulds were removed after one day, but the specimens were not disturbed until the second day after

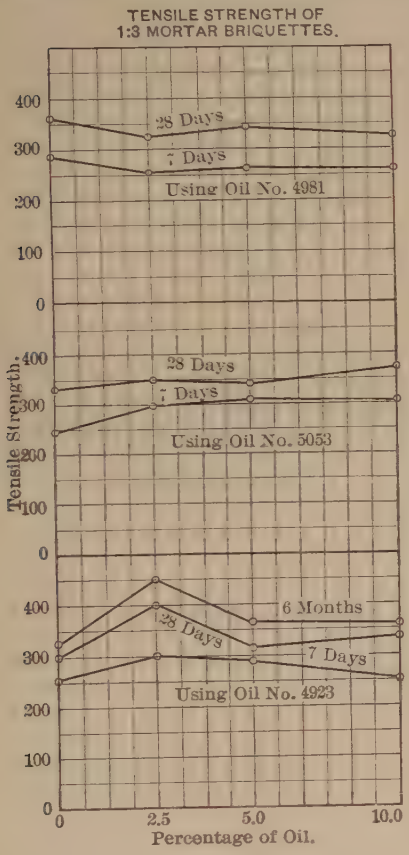


Fig. 1.

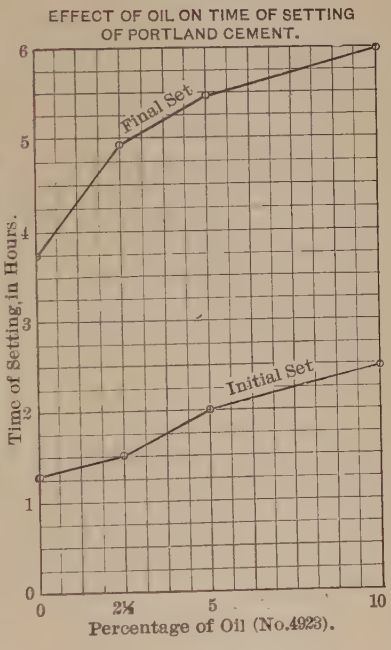


Fig. 2.

moulding. Air-cured specimens were kept moistened for 7 days after moulding. All tests were made in a universal machine having a capacity of 200 000 lb. The bearing surfaces of all mortar specimens were bedded in several thicknesses of blotting paper. It was found necessary to bed the top surface of each concrete specimen in plaster of Paris before testing, blotting paper being used in addition to this

bedding. The specimens were crushed at a speed of 0.152 in. per min., and the crushing load was taken at the drop of the scale beam.

The numerical results of the crushing tests are given in Table 5, and are plotted on Figs. 3 and 4. As would be expected with an inert substance like oil, the crushing strength decreases in general with increasing quantities of oil. In the case of the oil-mixed mortars, in only one instance is there a serious decrease of strength with the

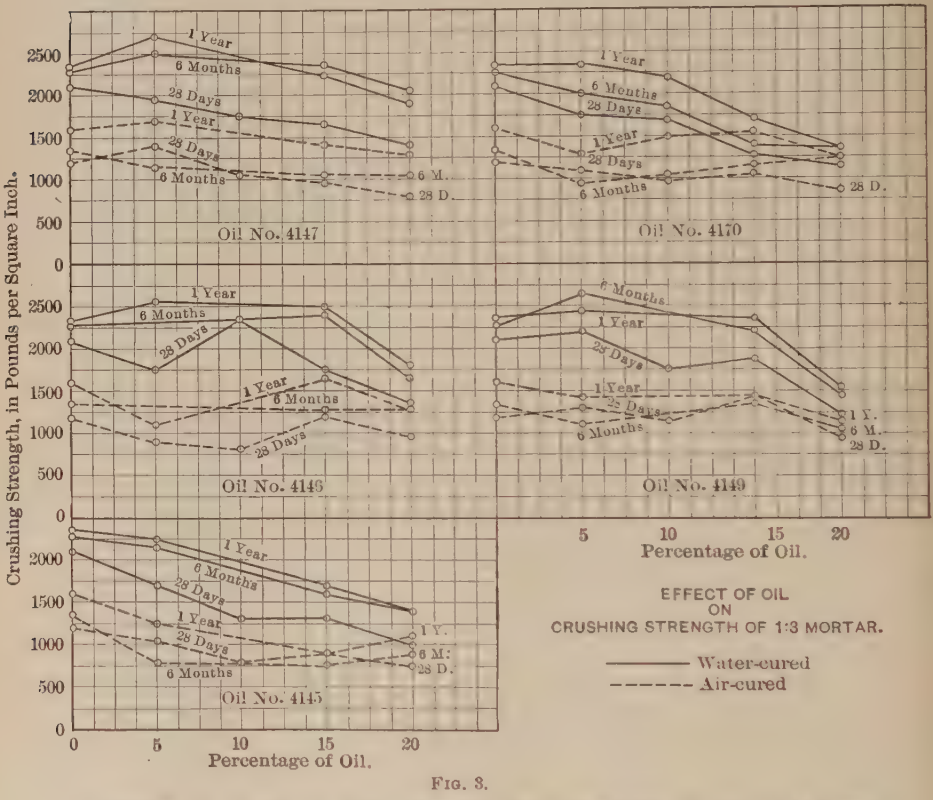


FIG. 3.

addition of oil up to 10%, namely, that in which oil No. 4 145 was used. At 28 days water-cured mortar specimens containing 10% of oil are at least 80% as strong as the plain mortar specimens. In every instance there is a substantial gain in strength at 6 months over that at 28 days; this is equally true of the one-year specimens. Specimens cured in air are somewhat erratic in behavior. In some instances a considerable gain in strength occurs, in others, a noticeable decrease.

TABLE 5.—Tests of Oil-Mortar and Oil-Concrete Cylinders.

Crushing Strength, in pounds per square inch.

All 1 : 3 mortar (by weight) except specimens 6c to 18c.

Specimens 6c to 12c are of 1 : 3 : 6 Concrete; Specimens 13c to 18c are of 1 : 3 : 5 Concrete (by volume).

The numbers in parentheses indicate the number of specimens tested.

Mark.	Percentage of oil.	Character of oil.	Stored in air.			Stored in water.		
			28 days	6 months.	1 year.	28 days.	6 months.	1 year.
No.								
1	0		1 170 (5)	1 350 (2)	1 597 (2)	2 135 (6)	2 275 (2)	2 308 (2)
17	5	Fluid Residual Oil No. 4 145	1 050 (2)	830 (2)	1 220 (2)	1 715 (2)	2 160 (2)	2 240 (2)
2	10	" " " " "	790 (1)	.....	.....	1 320 (1)	.....	.....
22	15	" " " " "	895 (2)	780 (2)	880 (2)	1 290 (2)	1 615 (2)	1 700 (2)
27	20	" " " " "	775 (2)	910 (2)	1 125 (2)	1 020 (2)	1 420 (2)	1 425 (2)
18	5	Fluid Residual Oil No. 4 146	915 (2)	.....	1 110 (2)	1 770 (2)	.....	2 560 (2)
3	10	" " " " "	830 (1)	.....	.....	2 400 (2)	.....	.....
23	15	" " " " "	1 215 (2)	1 275 (2)	1 670 (2)	1 775 (2)	2 410 (2)	2 475 (2)
28	20	" " " " "	935 (2)	1 295 (2)	1 380 (2)	1 345 (2)	1 640 (2)	1 780 (2)
19	5	Fluid Residual Oil No. 4 147	1 405 (2)	1 155 (2)	1 715 (2)	1 965 (2)	2 475 (2)	2 700 (2)
4	10	" " " " "	1 070 (1)	.....	.....	1 720 (1)	.....	.....
24	15	" " " " "	975 (2)	1 085 (2)	1 435 (1)	1 670 (2)	2 350 (2)	2 250 (1)
29	20	" " " " "	795 (2)	1 055 (2)	1 310 (2)	1 400 (2)	2 010 (2)	1 895 (2)
21	5	Fluid Residual Oil No. 4 149	1 305 (2)	1 105 (2)	1 395 (2)	2 220 (2)	2 655 (2)	2 450 (1)
6	10	" " " " "	1 170 (1)	.....	.....	1 760 (1)	.....	.....
26	15	" " " " "	1 440 (2)	1 395 (2)	1 330 (2)	1 895 (2)	2 215 (2)	2 320 (2)
31	20	" " " " "	925 (2)	1 085 (2)	1 135 (2)	1 230 (2)	1 475 (2)	1 530 (2)
7	5	Cut-back Oil Asphalt No. 4 170	1 010 (2)	940 (2)	1 315 (2)	1 760 (2)	2 065 (2)	2 345 (1)
8	10	" " " " "	980 (2)	1 005 (2)	1 510 (2)	1 680 (2)	1 825 (2)	2 200 (1)
9	15	" " " " "	1 055 (2)	1 170 (2)	1 555 (2)	1 265 (2)	1 365 (2)	1 685 (1)
10	20	" " " " "	890 (2)	1 220 (2)	1 825 (2)	1 175 (2)	1 340 (2)	1 375 (1)
6c	0		1 020 (3)	.....	.....	1 370 (2)	.....	.....
7c	2½	Fluid Residual Oil No. 4 923	750 (2)	.....	.....	1 253 (3)	.....	.....
8c	5	" " " " "	711 (2)	.....	.....	1 142 (2)	.....	.....
9c	10	" " " " "	656 (2)	.....	.....	961 (3)	.....	.....
10c	15	" " " " "	542 (2)	.....	.....	750 (2)	.....	.....
12c	25	" " " " "	605 (3)	.....	.....	620 (3)	.....	.....
13c	0		920 (2)	.....	.....	1 342 (3)	.....	.....
14c	2½	Fluid Residual Oil No. 4 923	787 (2)	.....	.....	1 250 (2)	.....	.....
15c	5	" " " " "	720 (2)	.....	.....	1 082 (2)	.....	.....
16c	10	" " " " "	698 (2)	.....	.....	1 040 (2)	.....	.....
17c	15	" " " " "	693 (2)	.....	.....	1 040 (2)	.....	.....
18c	20	" " " " "	572 (2)	.....	.....	904 (2)	.....	.....

Concrete, like mortar, when mixed with oil, shows a decrease in strength, that containing 10% of oil having roughly 75% of the strength of plain concrete when stored in water. Air-cured concrete, mixed in proportions of 1:3:6 and 1:3:5, containing 10% of oil, has 65 and 75%, respectively, of the strength of the plain concrete mixtures. At the completion of the one-year tests on compression, a

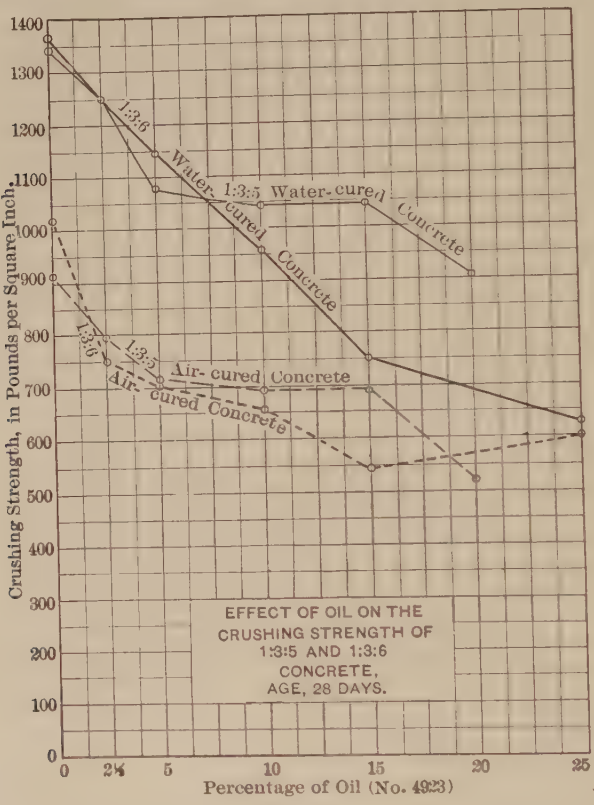


Fig. 4.

marked increase in strength is shown over similar tests at 6 months. The next tests will be made at the expiration of two years.

Toughness.—Cylinders of concrete, 6 in. in diameter and 6 in. high, were tested in the Page impact machine under the blows of a 10-kg. hammer, falling upon a 5-kg. plunger from successively increasing heights of 1 cm. The height of the last blow, causing rupture, corresponds to the number of blows. The end of the plunger in contact

with the test pieces is spherical in shape, with a radius of 3 cm. The specimens were set in plaster of Paris on a piece of plate glass, and after the plaster had hardened they were mounted in the impact machine, the bedded end on the anvil. Fig. 1, Plate XXXIII, is a view of the impact machine, and the specimen after failure. The results of this test indicate that the toughness or resistance to impact of oil-mixed concrete differs but slightly from that of plain concrete. The impact tests were made on only one specimen from each mix. Failure was indicated by the development of radial cracks, of which there were sometimes two and as often three. The numerical results of the impact tests are given in Table 6.

TABLE 6.—Number of Blows Required to Produce Failure.

Percentage of oil, No. 4 923.	1:3:5 Concrete.		1:3:6 Concrete.	
	Air-cured.	Water-cured.	Air-cured.	Water-cured.
0	18	15	15	21
2½	15	33	14	18
5	23	20	15	20
10	13	20	12	21
15	14	20	13	16
20	10	22	..	..
25	..	..	15	16

Modulus of Elasticity.—Tests for the modulus of elasticity were made on specimens of the usual size, 8 in. in diameter and 16 in. high. The deformations were measured with a double micrometer compressometer,* shown in Fig. 2, Plate XXXIII. The specimens were carefully bedded in plaster of Paris, a spherical bearing block being interposed between the top of the specimen and the moving head of the machine. Loads were applied in 2 500-lb. and 5 000-lb. increments, and were released to the initial load of 500 lb. after each increment of 5 000 lb., thus permitting of the determination of the amount of permanent set.

Typical stress-deformation and permanent-set curves are shown in Figs. 5 and 6. The values of the initial modulus of elasticity are given in Table 7.

* This instrument is described by J. M. Porter, Assoc. M. Am. Soc. C. E., in *Proceedings*, Am. Soc. for Testing Materials, Vol. X, 1910.

TABLE 7.—Initial Modulus of Elasticity—Age, 28 Days.

Percentage of oil, No. 4 923.	1:3:5 Concrete.		1:3:6 Concrete.	
	Air-cured.	Water-cured.	Air-cured.	Water-cured.
0	1 300 000	2 550 000	1 300 000	2 200 000
2½	1 350 000	2 700 000	1 000 000	2 400 000
5	1 250 000	2 350 000	850 000	1 900 000
10	1 700 000	3 950 000	1 150 000	1 900 000
15	1 300 000	2 500 000	730 000	2 050 000
20	1 400 000	3 000 000	.....	.....
25	.....	.....	1 400 000	1 950 000

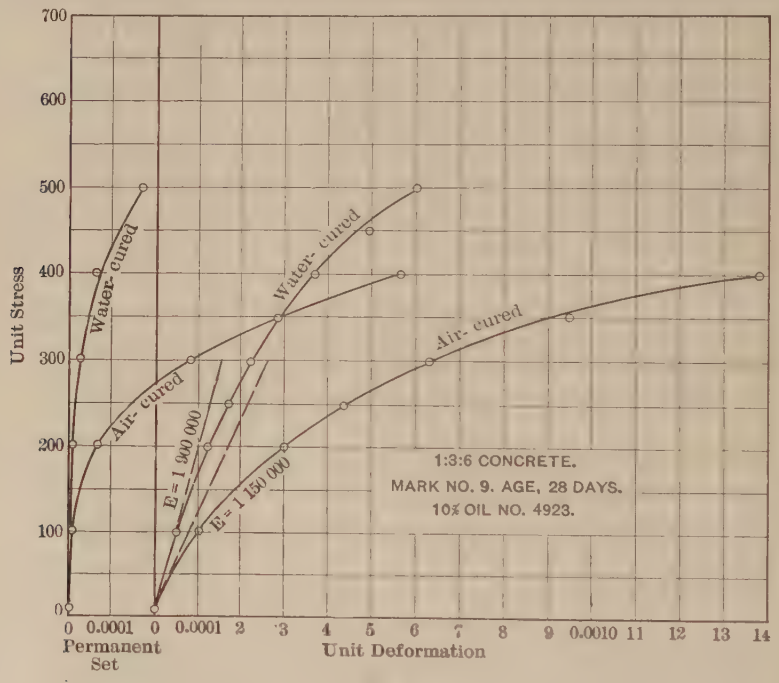


Fig. 5.

The results in Table 7 indicate that the addition of oil to concrete has very little effect on its stiffness, the initial modulus of elasticity, with a few exceptions, showing but little change. The effect of storing in water is to increase the modulus of elasticity to more than that of air-cured specimens, and is as true with oil as without it.

Permanent Set.—In the endeavor to discover the effect of oil on the elasticity of concrete, micrometer readings of permanent set were taken

FIG. 1.—TOUGHNESS TEST.

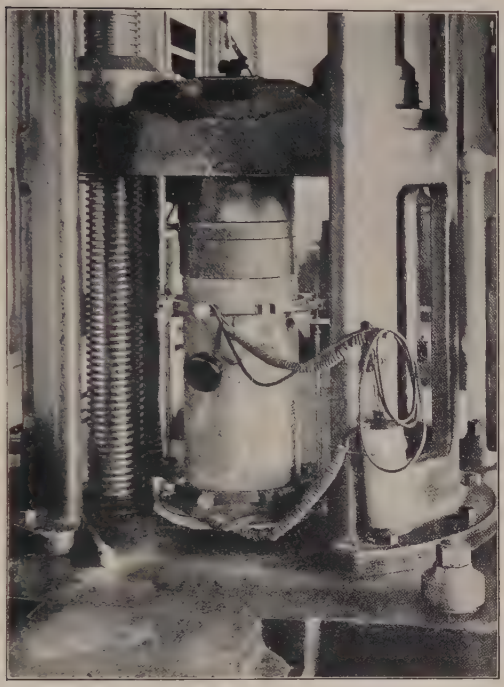


FIG. 2.—TEST FOR MODULUS OF ELASTICITY.

FIG. 3.—ABSORPTION TEST.

after releasing the load to its initial value of 500 lb. In 1:3:5 concrete, containing as much as 10% of oil, the permanent deformation existing after the application of 500 lb. per sq. in. was less than that in the specimen containing no oil. This was true of both air-cured and water-cured specimens. The elasticity of 1:3:6 concrete appears to suffer with the addition of oil, the permanent set of the 10% oil mixture, air-cured, being 0.00076 in. per inch of length, as against 0.00019 in. for the plain concrete specimens. The corresponding ratio

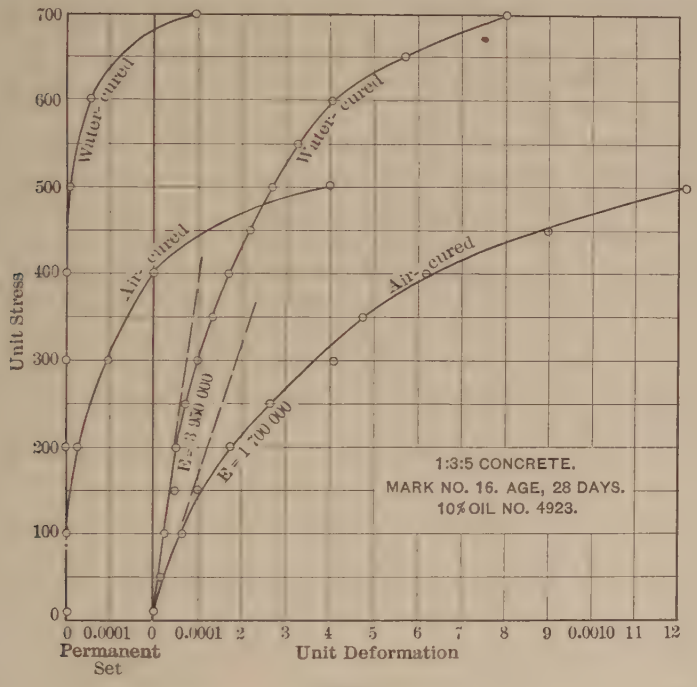


FIG. 6.]

appears to exist for the water-cured specimens. The permanent set of the 10% oil mixture was 0.00017 in. after the application of 500 lb. per sq. in. Here, again, the beneficial effect of storing in water is well marked, the permanent set being very small for water-cured concrete, when compared with that cured in air.

Absorption.—In order to determine the water-proofing qualities of oil-mixed concrete, compared with those of a plain mixture, absorption tests were made on 1:3:6 concrete cylinders, 6 in. in diameter and

6 in. high. The cylinders were dried to constant weight in an oven, after being cured for 15 days in air. They were then completely immersed in water, and were weighed every day until they had attained constant weight. The results of these tests are plotted on Fig. 7. It will be noted that the addition of oil decreases greatly the percentage of absorption, the cylinder containing 10% of oil absorbing but 1.7% of water, based on the dry weight, while 6.25% was absorbed by the cylinder containing no oil. In almost every case the maximum absorption was attained within one day after immersion, and, furthermore, the rate of absorption seemed to be inversely dependent on the quantity of oil added. It will be noted that there is approximately only 0.5% difference between the absorption of the cylinders containing, respectively, 10 and 25% of oil. The inference to be drawn from this fact is that 10% of oil is nearly as effective as 25%, as far as water-proofing is concerned.

Four vessels of 1:3 mortar, each 2½ in. high, about ½ in. thick, and having an outside diameter of 8 in., after hardening in moist air for one week, were immersed in water to a depth of 5 cm. Vessel No. 1 was made with no admixture of oil. A damp spot appeared inside about 1 min. after immersion. In one hour it was completely water-soaked, the moisture having climbed by capillarity above the level of the surrounding water. Within a few days after immersion, water had penetrated the plain mortar vessel until its level inside was the same as that outside. Vessels Nos. 1 and 3 are shown on Fig. 3, Plate XXXIII. Vessels Nos. 2, 3, and 4 contained, respectively, 5, 10, and 20% of oil. After an immersion of 6 months, the interiors of the oil-mixed vessels are as dry as they were on the day they were placed in the water. A corresponding series of tests was made on mortar vessels, made to contain water, with like results. The plain mortar vessel leaked, and those of oil-mixed mortar were perfectly tight. These results show with certainty the water-proofing qualities of oil-mixed mortar and concrete.

Permeability.—The following tests were made on specimens of 1:3 mortar, 3 in. thick and 6 in. in diameter: Plain 1:3 mortar, at the age of 28 days, under a pressure of 30 lb. showed dampness in about ½ hour. Under a pressure of 40 lb., applied for 24 hours, the leakage amounted to 146 cu. cm. Specimens of the same mixture, but con-

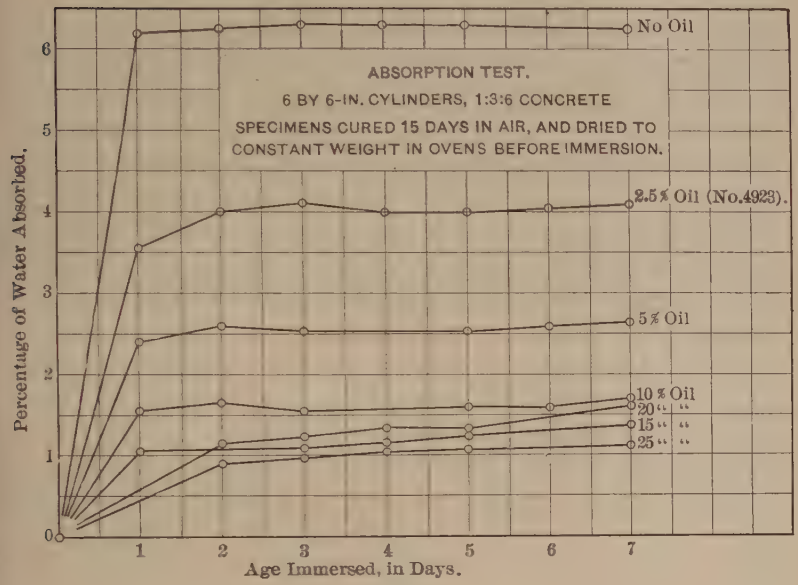

ABSORPTION TEST.
6 BY 6-IN. CYLINDERS, 1:3:6 CONCRETE
SPECIMENS CURED 15 DAYS IN AIR, AND DRIED TO CONSTANT WEIGHT IN OVENS BEFORE IMMERSION.
No Oil
2.5% Oil (No.4923).
5% Oil
10% Oil
20 " "
15 " "
25 " "
Percentage of Water Absorbed.
Age Immersed, in Days.

FIG. 7.

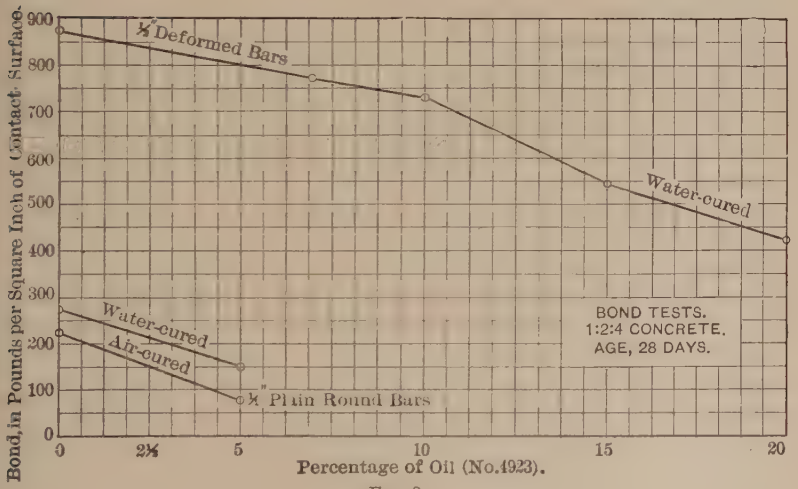

½" Deformed Bars
Water-cured
Water-cured
Air-cured
½" Plain Round Bars
BOND TESTS.
1:2:4 CONCRETE.
AGE, 28 DAYS.
Bond, in Pounds per Square Inch of Contact Surface.
Percentage of Oil (No.4923).

FIG. 8.

taining 5 and 10% of oil No. 4 923, remained perfectly dry for 24 hours, under a pressure of 40 lb.

A ½-in. coat of 1:3 mortar, containing 10% of oil No. 4 923, applied to the surface of a very porous concrete, was absolutely effective in preventing all leakage, even under comparatively high pressure.

Bond Tests.—Tests were made to determine the relative strength in bond of oil-mixed and plain concrete. Specimens were made of 1:2:4 concrete, mixed with 23½% of water, based on the weight of cement and dry sand, making a rather wet mix. Rods 12 in. long were embedded in the center of cylinders 8 in. in diameter and 8 in. long, one end projecting about ½ in. through the bottom. The specimens were bedded in plaster of Paris on a cast-iron plate mounted on a spherical bearing block. The test consisted in pushing the rods through the concrete, the point of failure being taken at the drop of the scale beam.

Two kinds of bars were used, plain and deformed. All specimens were tested at 28 days, and the results are plotted on Fig. 8. The points plotted represent the average of two tests, which in all cases ran very closely together. The results show that the effect of oil is to decrease the bond strength, the decrease depending on the quantity of oil added. It is evident that the bond between plain bars and concrete is affected so seriously by the admixture of oil that it would be extremely inadvisable to use such a combination. The bond of deformed bars is not affected so seriously. With an admixture of 10% of oil, which is as much as is necessary to use, the bond strength is 84% of that attained by the mixture containing no oil.

Service Tests.—During April and May, 1910, two bridge surfaces of oil-concrete were laid at Ridgewood, N. J. In the Borough of Richmond, New York City, during May and June, 1910, about 400 ft. of street surface was laid with different aggregates. About 300 ft. of street in Washington, D. C., was surfaced in June, 1910. Numerous inspections of this work have been made, and, up to the present time, it is all in very good condition. About ½ mile of roadway, laid with 10% oil-concrete in the suburbs of Harrisburg, Pa., in the summer of 1910, is also reported to be in very good condition. A vault, 112 ft. long and 18 ft. wide, in the United States Treasury Department, was built in the autumn of 1910. The side-walls of this vault contain 10% of oil. Its roof was constructed of ordinary reinforced concrete with

about 3 in. of 10% oil-concrete on top. For several months this roof was subjected to a head of water of several feet without showing any signs of leakage. Another vault, in the north end of the Treasury, on account of leakage, has never been available for storing anything of value. Its roof was treated with oil-concrete, and is now perfectly dry. Numerous floors in the sub-basement of the Treasury Department, and a floor in the Office of Public Roads, have been constructed with 10% oil-concrete, and have remained absolutely free from dampness. A tank, about 12 ft. long, 4 ft. wide, and 4½ ft. high, constructed in the Office of Public Roads during the winter of 1910, has been continually full of water, and has not shown the slightest signs of leakage.

Summary of Conclusions.—The following tentative conclusions as to the effect of the oils used in cement and concrete may be drawn from the foregoing investigations:

1. The tensile strength of 1:3 oil-mixed mortar differs very little from that of plain mortar, and shows a substantial gain in strength at 28 days and 6 months over that at 7 days.

2. The times of initial and final set are delayed by the addition of oil, 10% of oil increasing the time of initial set by 90%, and the time of final set by 60 per cent.

3. The crushing strength of mortar and concrete is decreased by the addition of oil to the mix, concrete with 10% of oil having roughly 75% of the strength of plain concrete at 28 days. At the age of one year the crushing strength of 1:3 mortar suffers but little with the addition of oil in quantities up to 10 per cent.

4. The toughness or resistance to impact is affected but slightly by the addition of oil in quantities up to about 10 per cent.

5. The stiffness of oil-mixed concrete appears to differ but little from that of plain concrete.

6. The results of tests for permanent deformation do not indicate that a law is followed by oil-mixed concrete.

7. Oil-mixed mortar and concrete containing 10% of oil have very little absorption, and, under low pressures, both are water-proof.

8. Oil-mixed mortar containing 10% of oil is absolutely water-tight under pressures as high as 40 lb. per sq. in. Tests indicate that oil-mixed mortar is effective as a water-proofing agent when plastered or painted on either side of porous concrete.

9. Bond tests show the inadvisability of using plain bar reinforce-

ment with oil-concrete mixtures. With deformed bars the bond is not weakened seriously by the addition of oil in quantities up to 10 per cent.

A patent has been granted to the writer for mixing oil with Portland cement concrete and hydraulic cements giving an alkaline reaction. All citizens of the United States are at liberty to use this process without the payment of royalties.

The writer takes this opportunity to express his thanks to A. S. Cushman, Assoc. Am. Soc. C. E., and Mr. A. T. Goldbeck for their most valuable assistance in conducting these experiments.

DISCUSSION.

Mr. Newton.

S. D. Newton, Assoc. M. Am. Soc. C. E. (by letter).—It seems to the writer that Mr. Page's experiments on oil-mixed mortar and concrete, fragmentary and incomplete as he himself confesses them to be, open up a very useful field of knowledge to the engineer, and go far toward solving the problem of water-proofing concrete structures. The possibilities of further experimentation are almost unlimited, and it is to be hoped that some one with ample funds and facilities at his command will take hold of the matter at once. As the first credit undoubtedly belongs to Mr. Page, all will agree that he is the one to follow the matter up, if it can be arranged; and the writer would be in favor of memorializing Congress to make an appropriation placing funds at the disposal of the Public Roads Department for this purpose.

Mr. Page's conclusions are based on experiments with some eight samples of mineral oils, and, for each test, his results are based on breaking only one or two specimens of concrete or mortar mixed with each oil. It is not likely that the breaking of a larger number of specimens would alter the results materially, but, nevertheless, the conclusions would have been more convincing if three or more times the number of tests had been made with each of these individual oils. Also, if a wider range of oils and concrete materials had been used, it would have been possible to determine closely what kind or class of oil is most efficient and cheapest in the long run for the purpose, as, for instance, between oils having a paraffin or an asphaltic base, and of different degrees of refinement.

It is generally conceded that animal oils tend to disintegrate concrete; but, as far as the writer knows, this conclusion is based on repeated external applications of such oils (as, for instance, spigot droppings of "signal oil" on concrete floors), and not on studies of oil-mixed concretes, such as those made by Mr. Page. Experiments along this line would be highly illuminating, as would also experiments with linseed, cottonseed, and other vegetable oils incorporated in concrete. Ferro-Portland cements are coming into use for structures subject to the action of salt water; it would be interesting to know in what way, if at all, oil-mixing would affect them, and whether the results would differ from those which obtain with ordinary cements.

A series of long-time tests (extending over two or three years or more) should be started at once to determine whether or not the water-proofing effect of oil-mixing is permanent, and what proportion of the observed decrease in crushing strength, as between oil-mixed and ordinary concrete, remains at the end of the longest period of the test. Accelerated tests could, perhaps, be devised in order to give at once some little knowledge as to the permanence of the water-

Mr. Newton. proofing. Tests to determine the chemical reactions are also desirable. It is probable that a certain part of the cement unites with the oil to form an emulsion, but why this should not decrease the tensile strength (for instance) in something like the same proportion as it does the crushing strength, is a point which has not yet been made clear.

As to the service tests described by Mr. Page, these, too, are instructive, but as none of the structures mentioned is yet 18 months old, it is too early to draw sweeping conclusions as to their ultimate behavior. The writer has had some little experience along the line of concrete pavements, and he must confess that he is skeptical as to the advantage of oil mixtures for that purpose. His observations go to show that it is not so much the absorbing power of the concrete which causes failure, as it is the cracking due to contraction during cold weather; nor can he see how an admixture of oil would tend to add anything to the elasticity of the surface or make the pavement easier on draft animals.

Mr. Page's action in throwing his patent open to the use of the general public cannot be commended too highly, and the writer believes that all inventions, of use to engineers, should be treated in the same way.

Mr. Oxholm. T. S. Oxholm, M. Am. Soc. C. E. (by letter).—In the latter part of 1909, the writer met Mr. Page at a convention of cement manufacturers, and became interested in the paper which he read at that meeting describing certain experiments which he had made with mortar mixed with oil.

An offer was subsequently made by the President of the Borough of Richmond to the United States Department of Public Roads to experiment with oil concrete under Mr. Page's direction, as it was understood that that Department had no funds with which to carry on experiments of this character.

In the summer of 1910 arrangements were completed, and a stretch of pavement, of about 1 500 sq. yd., was laid on Innis Street, Borough of Richmond. This piece of pavement is referred to in the paper, and as the experiment was made under the joint control of Mr. Page and the writer, a few details may be of interest. A number of tests of briquettes of this material were made in the laboratory of the President of the Borough of Richmond, which, compared with the tests made by the author, agreed remarkably well, and indicated that the proper quantity of oil would be 15% of the weight of the cement.

When the mixing was first started, an attempt was made to follow the method recommended by the Department of Public Roads, viz., that the cement and sand should be placed in the mixer and enough water added to make a stiff mortar, after which the oil should be added and mixed, then the stone placed in the machine, and the whole mass thoroughly combined. A "Ransome" batch mixer was used

for the concrete, but, in attempting to follow the method outlined, it was found that the oil stuck to the surface of the mixer, and would not combine thoroughly with the mortar. Mr. Oxholm.

The method finally adopted was to place the sand and cement in the machine, and, after mixing, add as much water as was necessary to bring the complete batch to the proper consistency. The quantity of water was determined by trial, and was sufficient to make a thin mortar. Then the oil was added, the dampened stone was dumped into the mixer immediately after the oil, and the whole mass was thoroughly mixed. With this method the oil quickly emulsified and combined intimately with the other materials.

The following is the report of Louis L. Tribus, M. Am. Soc. C. E., Consulting Engineer and Acting Commissioner of Public Works, to the President of the Borough, describing this experimental pavement, and including the reports of the assistant engineer, and the engineer in charge of the work:

"Pursuant to your authorization from time to time, we have experimented with every class of road pavement which seemed to promise to be of service in the Borough of Richmond and to meet different conditions of grade, exposure, traffic, and development of the adjoining property.

"A year ago last spring it was deemed desirable to lay a specimen pavement of 'oil concrete.'

"The United States Director of Public Roads was interested in the subject, and assigned an engineer to be present during the laying of pavement which, except for a very small sample laid on a bridge in Ridgewood, N. J., was to be the first demonstration of the kind in the United States.

"The following reports, filed respectively by W. B. Grubbe, Asst. Engineer, in direct charge of the test, and Theodor S. Oxholm, M. Am. Soc. C. E., Engineer in charge of construction generally, give the salient points as to the original laying of the pavement and its condition after nearly fifteen months of service:

"'JULY 20, 1911.

"'Attached is a map of Innis Street, showing the limits of the various mixtures of oil concrete pavement laid in May, 1910.

"'The concrete was mixed in the proportion of one part cement, 2 parts sand, and 4 parts broken stone, ¾ in., with the addition of oil at the rate of 15% of the weight of the cement. With the Texaco oil an allowance was made for a 20% cut back.

"'The cement and sand was mixed and water added to form a mortar, after which the oil and then the stone was added. The whole mass was then thoroughly mixed and deposited on the street to form a wearing surface 4 in. in thickness. A mechanical mixer was used for the concrete. After spreading, the concrete was tamped, and after the initial set, was covered with sand.

"'I have made a recent examination of the pavement and found that the joint at the end of each day's work is very marked. The

Mr. Oxholm.

second day's work on the section laid with Vulcanite cement and Standard oil is fairly good, and also the 3rd. On the 4th day's work there are a number of small depressions, caused originally by throwing sand on the pavement before it was sufficiently set.

"'In the section laid with Giant cement and Standard oil, there are a great many small cracks.

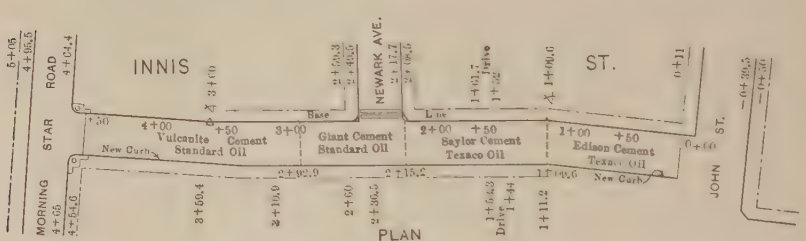
"'In the section laid with Saylor cement and Texaco oil, a portion of the surface seems to be powdery. In this portion the surface was struck with a straight edge and floated.

"'The section laid with Edison cement and Texaco oil, has a few cracks in the first portion; the latter portion is fairly good.'

"'SEPTEMBER 13, 1911.

"'Herewith please find plan and report of Assistant Engineer in charge of the experimental work of repaving Innis Street, between John Street and Morning Star Road, with oil concrete pavement a little over a year ago.

"'The general effect of the pavement is poor, and this is largely due to the lack of knowledge as to methods of construction, this being the first piece of work of this character ever attempted in this country, except a very small sample in Ridgewood, N. J., on a bridge. It is believed that a much better piece of work than this could be done with the knowledge gained, both as to the materials to be used and the methods of laying and finishing each day's work. I do not think, however, that this class of pavement, after being 15 months in use, has shown sufficient merit to make it advisable for us to lay any more of it at the present time.



PLAN
OF
AN EXPERIMENTAL PAVEMENT
OIL CONCRETE
LAID ON INNIS STREET
BOROUGH OF RICHMOND.

FIG. 9.

"'It will be remembered that this work was done under the supervision of the U. S. Bureau of Public Roads, Department of Agriculture, and an Engineer from that Department was at all times present on the work. The cement and oil were presented free of charge to the City for the purpose of experimentation, the City furnishing the other materials and a contractor doing the work under the supervision of this Bureau.'

"The sketch plan [Fig. 9] shows Innis Street with the general limits of the four sample pavements laid; one portion each being mixed with oil and Vulcanite, Giant, Saylor, or Edison cement.

"The grade of the street is very moderate (1.4%), just sufficient for good storm flow. Mr. Oxholm.

"It may be possible with a more complete knowledge of the characteristics of road oils and certain brands of cement that success may yet be secured in pavements of such character; but we do not see from the test made sufficient promise of value to feel disposed to recommend any further work along that line here."

For the information of any one desiring to inspect this pavement, it may be easily reached by the New York Ferry to St. George, and the North Shore train to Elm Park Station; Innis Street is one block south of the station.

Lars R. Jorgensen, Assoc. M. Am. Soc. C. E. (by letter).—For some reason or other the writer had thought that oil and concrete would not work well together. The tests made by Mr. Page, therefore, were a surprise to him, as they seem to show conclusively that, within certain limits, the oil does not weaken the concrete, or only affects it to a small extent. Mr. Jorgensen.

The practical applications of this oil-mixed concrete may be very extensive. For the writer, it has most interest when used in dam construction for the purpose of making a tight dam. A few feet next to the up-stream face built of oil-mixed concrete should act as a good water-proofing. To build the whole structure of oil-mixed concrete would not seem to be practical, because, in order to keep the value of the modulus of elasticity high with oil-mixed concrete, it would require a richer mixture than ordinarily used (if future tests on completely aged specimens do not show different results); besides, the oil adds to the cost, and makes the concrete slow-setting—too slow for many cases of dam building, where the load must be taken up almost as soon as the structure is completed.

When used in connection with arch dams, it is of great advantage that the permanent set of oil-mixed concrete (not the possible shrinkage due to setting, but the permanent deformation after the reservoir has been filled once) is less than that of ordinary concrete. This is seen to be true for 1:3:5 concrete or richer, and a leaner mixture should not be used for a water-tight face. When the reservoir is empty, after having been filled at least once, and no vertical cracks have appeared on the arch, it is clear that, if the permanent deformation is smaller for the lining of oil-mixed concrete on the up-stream face than for the ordinary concrete in the remainder of the dam body, axial compression will exist along the up-stream face, tending to close any cracks which otherwise might open, even if the remainder of the structure is not stressed, or is a little in tension.

Why just the opposite is the case for a 1:3:6 mixture, and the permanent deformation of oil-mixed concrete is larger than for ordinary concrete (according to the tests, four times larger), is some-

Mr. Jorgensen.

what of a mystery. There may be a point where conditions reverse, but this seems to take place too abruptly. Possibly the specimen used was too green (28 days) considering the leaner mixture, but, if this proportion will still hold for completely aged concrete, it simply goes to show that oil-mixed concrete leaner than 1:3:5 cannot be used to advantage for dam facings. It will be interesting when tests have shown the relative value of the permanent deformation of oil-mixed and plain concrete for specimens from 2 to 3 years old, that is, after complete aging. Possibly then the law followed by the different mixtures in regard to permanent deformation may be found, from which information can be obtained as to what mixtures should not be used for certain structures.

Mr. H. de B. Parsons.

H. de B. Parsons, M. Am. Soc. C. E. (by letter).—The writer is interested in this paper, especially as he used oil-mixed mortar in the erection of a building in 1905. The building was much exposed to the weather and winter storms, and was constructed of brick and wood. Where the different kinds of construction joined, copper flashing was used.

In order to make tight joints between the vertical edges of the copper flashing and the brickwork, he used Portland cement mortar mixed with oil. A coating of this oil-mixed mortar was placed on the brickwork, and, while soft, the copper sheets were laid over it and fastened. Then the cement was forced under the edges of the copper in order to caulk any opening that might be left. Although exposed to the north, east, and west, these joints have remained tight.

The builder who did the work, stated that on previous jobs he had used oil in mixing cement mortar in order to make effective water-tight joints.

Mr. Phelan.

Frank J. Phelan, Esq. (by letter).—Logical engineers build for the future, and incline to the use of concrete on account of its greater commercial economy and permanency. The chief objection to its use for many purposes is its well-known porosity and capillarity. Below grade, a concrete wall becomes water-soaked, and, if above grade, under the influence of the elements, efflorescence appears on the surface and changes its appearance, thus condemning it artistically. To control this condition there is no remedy except to water-proof the mass by an integral method, and this is the aim of this paper.

Is mineral oil a safe and logical water-proofer? A mixture of 1:3, composed of 100 lb. of cement, and 300 lb. of sand, requires 70 lb. of water to temper the mortar properly. According to the standards, this mass when dry weighs only 425 lb., showing that the cement in crystallizing takes up in combination only 25 lb. of water, and the remainder (45 lb.) evaporates, leaving 10% of cell space, which water will again enter if supplied. Concrete mixes are some-

Mr. Phelan.

times graded to show 25% absorption. This problem, however, the busy architect or engineer is asking the chemist to solve by suggesting a water-proofing unit to mix with cement which will compensate for this physical action, to the end that, under the operation of a natural law, the capacity of the cement for absorbing moisture will be changed from positive to negative, so that water will not again penetrate the finished wall; and they naturally stipulate, further, that the tensile strength of walls thus treated must not be diminished. A foreign substance, whether it does or does not water-proof, is not admissible, but a water-proofing unit composed chiefly of the principal elements of cement, calcium base, which when added in small quantity will not arrest or interfere with the natural cementitious processes, but will form a basic insoluble compound in itself, is admissible; and it is such a unit that architects and engineers are expecting cement chemists to place at their command. This, if it could be secured, is an ideal, and will meet the objection so generally heard as to the use of cement for residential purposes. Compare such a standard with the use of mineral oil in the ratio of 10% of the weight of the cement, and at first flush it would seem that this patented method is comparatively worthless, for the following reasons:

1st.—It hurts the tensile strength of concrete, and this is admitted;
2d.—A 10% quantity adds too much to the cost of a cubic yard of concrete;
3d.—Its use is too dangerous and questionable, on work of any magnitude, where unskilled labor is a necessary factor;
4th.—It is a foreign substance;
5th.—It is a hydro-carbon—uncontrolled—unaffected by the alkaline elements with which it is mixed; out of harmony; penetrating in character in the absence of moisture; and hence, when diffused through the wet concrete, it will penetrate it after moisture evaporates, thus leaving an oil-soaked mass.

It water-proofs if it can be held in suspension, but this is doubtful. There are cases where oils spread over concrete surfaces gradually destroy the cement bond and cause disintegration. Hydro-carbon oil put with dry cement ruins it, as it kills crystallization. Its use can only be possible as a mix after the cement and other aggregates have been thoroughly manipulated with the tempering water, and even then it is admitted that the set and tensile strength are changed and lowered. If the water is not gauged accurately, and the mortar, instead of being stiff enough to hold the oil in suspension, is liquid, what will become of the oil? It will merely move with the water to the face of the forms, penetrate there and become lost, or find its way to a low point and run off. In a moderately stiff mortar for trowel use, manipulation with the trowel will cause the oil to follow the

Mr. Phelan.

suction and collect on the surface; this is true with any substance which rides the water when introduced into the cement—any insoluble or fixed physical unit.

Hydro-carbon, non-drying, non-saponifying oil, is a dangerous element in concrete, and, to secure such water-proof results as this paper describes, requires knowledge of conditions and careful equative handling by expert labor. In a 1:3 mix, oil equal to 10% of the weight of the cement is equivalent to 2½% of the mass; in a 1:3:6 mix, it is 1 per cent. If a batch mixer is turning out ½ cu. yd. of concrete or more every 2 or 3 min., where will the oil be when the forms are filled? It will certainly be on the top, or side, or at the point of least resistance.

To water-proof cement logically, the water-proofing unit must be diffused uniformly throughout the mass, and must remain there. The writer submits that it is irrational, illogical, and dangerous to introduce into cement this foreign substance, mineral oil.

Mr. Richardson.

CLIFFORD RICHARDSON, M. AM. SOC. C. E.—The speaker has followed with interest Mr. Page's experiments with Portland cement mortar and concrete mixed with oil since their inception. These experiments arose from the observations of Allerton S. Cushman, Assoc. Am. Soc. C. E., that mineral oil could be emulsified with lime-water very readily, and that of Albert Moyer, Assoc. Am. Soc. C. E., that if this were the case a similar emulsion could be obtained with a Portland cement paste.

The results obtained by Mr. Page are very striking, and, no doubt, of considerable importance in certain directions. That a Portland cement paste or mortar will emulsify so readily a considerable percentage of mineral oil, is undoubtedly due to the free lime which is liberated on mixing the cement with water. The emulsion is so complete that it is impossible to distinguish particles of oil except under the microscope, and there is no possibility of a separation of the particles, when once emulsified, as has been suggested by one of the discussors. There is no possibility of the destruction of the bond of the Portland cement by the mineral oil, although this would happen very readily if a vegetable or animal oil were mixed with concrete, owing to the decomposition of the latter and the liberation of free acid, which would attack the lime of the Portland cement and destroy its bond.

A question has been asked as to the character of the oil which is suitable for use in oil-mixed concrete. Any of the denser oils, preferably more or less asphaltic, which are supplied by the Standard Oil Company, the Sun Company, the Texas Company, the Indian Refining Company, and the Barber Asphalt Paving Company, will be suitable for the purpose. The cost of the oil is only a few cents per

gallon, and its use cannot add largely to the cost per cubic yard of concrete. Mr. Richardson.

To the speaker it seems that the principal use of oil concrete will be in the form of an application of the material over structures built of ordinary concrete and to act as a water-proofing material. It is also worthy of note that many colloids will emulsify mineral oils in the same way as Portland cement.

At the Second International Road Congress, held in Brussels in 1910, a material, known as sand-clay asphalt, was exploited by Mr. Tardres, Engineer of Bridges and Roads at Algiers, some of it being laid on the exhibition grounds as a road surface. It is produced, according to the inventor, by mixing 12 parts of clay with a sufficient quantity of water (about 2 to 6 parts), and adding, while mixing, 12 parts of tar and 100 parts of hard sand. The clay paste emulsified the bituminous material with the same ease as the Portland cement paste emulsifies the oil, and it was eventually found that the clay could be replaced by any other colloidal matter mixed with water, such as dextrin. Such a mixture, as observed by the speaker, forms a very plastic mass, and can be readily spread or floated into place. It dries out rather slowly, but eventually becomes quite hard. The inventor claimed that it could be used very economically for binding together broken-stone roads, for walks, for cellars and stables, and for similar purposes.

At the same Congress a material was exploited under the name of "Kiton," by Dr. F. Raschig, of Ludwigshafen-on-Rhine, Germany, which is apparently of the same type—an emulsion of clay and tar rather than oil. A considerable quantity of this material has been used in Germany for road construction.

Mr. Page's method of making oil concrete can be advantageously modified by replacing a portion of the cement with clay. This makes a much more plastic and impervious mortar, it being well recognized that the addition of clay to a concrete makes it more impervious. Experiments in this direction have shown that there is no difficulty in making such a mortar, and that, when properly proportioned, it sets to an extremely hard mass and has a decided advantage over Mr. Page's straight oil cement concrete, particularly because of its impermeability, plasticity, and the ease with which it can be floated into place.

R. D. Coombs, M. Am. Soc. C. E.—What are the merits of an oil mixture, as applied to the case of a beam or column continually exposed to moisture; a reinforced member with comparatively little concrete outside the reinforcement? Mr. Coombs.

In other words, would an oil-mixed concrete protect the reinforcement from corrosion (in case corrosion were possible), from moisture

Mr. Coombs.

entering the concrete by capillary action, assisted, perhaps, by small cracks due to bending, etc.?

To be more specific, assume that a concrete pole is located in a tidal marsh, where the concrete at the base is continually subject to the condition of alternate wetting and drying. Assume, further, that such a pole has 1½ in. of concrete outside the rods, that it may be subjected to more bending than usual in concrete structures, and that the design contemplates high unit stresses and concrete of a very high grade. Would there be any advantage in using the oil mixture in such a case?

Mr. M. G. Parsons.

MAURICE G. PARSONS, JUN. AM. SOC. C. E. (by letter).—The experiments recorded in the paper demonstrate that the old fear of the harmful effects of oil in concrete is not entirely justifiable. Mr. Page has carried the work to a point where the desirability of further research is evident. At present any discussion of the subject by others is likely to be rather of an inquisitive than supplemental nature. Four questions might be asked:

1. What effect on concrete will be produced by a change in the viscosity of any one oil?

2. How will oils of different viscosities influence concrete?

3. Will temperature, within practical working limits, produce, in and of itself, an appreciable result?

4. Can oils of widely different properties (such as crude, neat's foot, or machine) be relied on to give similar results?

TABLE 8.

Crushing strength of water-cured concrete containing 10% of oil, at 28 days.	Oil.		Remarks.
	Viscosity.	No.	
1 710 lb. per sq. in.	2.5	4 981	5.25 times the tensile strength.
2 400 " " " "	6.4	4 146	1 : 3 months.
1 320 " " " "	14.2	4 145	1 : 3 months.
1 840 " " " "	17.4	5 053	5.25 times the tensile strength.
1 720 " " " "	18.2	4 147	1 : 3 months.
1 750 " " " "	20.2	4 149	1 : 3 months.
1 740 " " " "	65.1	4 927	5.25 times the tensile strength.
			1 : 3 months.

These questions the writer cannot answer from a study of the paper, but they are fundamental in actual construction. Commenting on the first two, it may be remarked that:

1. The wide change, with temperature, in the fluidity of crude oil might give trouble.

2. An extremely fluid oil would possibly surround the cement particles and cause a serious loss of strength.

Mr. M. G. Parsons.

In view of the fact that the information regarding oil-mixed concrete is so incomplete, the writer would hesitate to use it without consulting Mr. Page.

Table 8, bearing on the foregoing questions, has been prepared from the information in the paper.

The factor, 5.25, in Table 8, was obtained by dividing the average crushing strength by the average tensile strength. The method of curing the tensile specimens is not known.

Mr. Thomes.

E. H. THOMES, ASSOC. M. AM. SOC. C. E.—From one of the discussions, the speaker received the impression that it was necessary to mix the sand, cement, and water together, and then add the oil and stone separately.

During the summer of 1911, the speaker had charge of experimental pavements constructed by the Borough of Queens, City of New York, in co-operation with the United States Office of Public Roads, on Hillside Avenue, in Jamaica, extending from the New York and Queens trolley tracks eastward to the Soldiers' Monument. Two sections of oil-cement-concrete pavement were laid on the old macadam foundation. The first section was about 100 ft. long, and 25 ft. wide, and was laid with 4 in. of 1:2:4 concrete mixed with "Standard" oil; the other section, of about the same length, was mixed with "Texaco" oil. A "Smith," hot, bituminous-concrete mixer with side-loader was used. In the first experiments the sand and cement were placed in the mixer, then water was added, then the oil and, finally, the wet stone. In the next experiments all the materials were placed in the mixer at the same time, and equally good results were obtained. It seems, therefore, that oil can be mixed in concrete without much additional expense, as all the ingredients can be incorporated at one time in a mechanical mixer. In both cases the oil seemed to mix thoroughly, and there was no apparent difference in the results.

The pavement was laid in August, 1911, and, at the present time (November, 1911), shows some wear in a few spots, especially at the junction of each day's work. Recently, "Tarvia X" and a layer of stone chips were spread over part of the worn spots, in order to determine the feasibility of repairing and maintaining the surface. This method of maintenance promises satisfactory results. The two sections of concrete pavement are separated by an expansion joint consisting of two courses of creosoted-wood blocks with asphalt-filled joints, and this treatment has given entire satisfaction. The wood contracts with the summer heat and expands with the winter moisture; the concrete is affected in the opposite way. This pavement is subjected to an average daily traffic of about 1 000 vehicles, mostly motor vehicles.

It is expected that the U. S. Office of Public Roads, will soon publish a detailed account of these experiments.

Mr. Dunham. H. F. DUNHAM, M. AM. SOC. C. E.—All comparative tests should be made with great accuracy and fairness, and a paper describing such tests should set out the various steps in close detail. For instance, it is well known that the thorough mixing of the cement and the aggregate is necessary, if it is desired to make a strong, impervious mortar or concrete. Now, if a certain amount of labor was expended in mixing the 1:3 mortar for one test and the same mortar was again worked over when oil had been added, and later subjected to a similar test, what assurance is there that the improvement shown was entirely due to the oil, and not in part to the more complete incorporation of the cement by the second mixing? With good cement and fairly well graded sand and gravel, a 1:3 mixture should be practically impervious to water without adding oil.

If the general intent of the experiments was to set forth the commercial value of a new substance or a new method, the effort seems to have been unwisely planned.

Oil as one ingredient must cost a certain sum delivered at the work. To introduce it in a given proportion as the work progresses is another item of expense, for no contractor would undertake to keep on hand a supply of oil and add a measured quantity to each mixer charge without being paid for the chances of delay, the extra time, and the extra labor. The estimated sum of these items should have been added to the cost of the oil, including the contractor's profit and the total sum expended for additional cement, A in quantity. Then a comparison between $1 + A + 3$ and $1 +$ oil $+ 3$ would give a result from which the commercial value of the oil method would be apparent. Although the extra quantity of cement would not be very large, it should be remembered that a small quantity often effects a marked change when added to that which is nearly sufficient to make a mortar impervious.

The reference to a patent is not clearly understood. Why should a patent be obtained if the method or process is to be used by all or any one? The author regrets his inability to make further helpful experiments. One function of a patent is to secure the incentive and the means to make further improvements, and, finally, to bring an article or a process to a high degree of perfection, in order that many may be benefited. The author's relation to the Government may have influenced his decision respecting his invention; but, to what extent should employes of the Government take patents from the Government and bestow them upon the people? The practice seems to be more in harmony with regal than with republican ideas.

Mr. Lippincott. J. B. LIPPINCOTT, M. AM. SOC. C. E. (by letter).—The writer submits Tables 9 and 10, containing the results of tests made with California crude asphalt oils having a gravity of 14° Baumé. These

Mr. Lippincott.

tests were made in the laboratory of the Los Angeles Aqueduct, at the cement mill at Monolith. The percentages of oil stated are in terms of the quantity of water added, and this quantity of water was 8% of the weight of the aggregates. The oil was added after the sand, cement, and water had been combined.

TABLE 9.—Sand Briquette Tests with California Crude Asphalt Oils. Briquettes Weathered Dry.

Briquette No.	Date made.	Percentage of water.	Tensile strength. 3 days.	7 days.	28 days.	3 months.	6 months.	Remarks.
1847	Feb. 9	8	185	245	265	260	230	5% Asphalt, crude, 14° Baumé
1848	" 9	8	155	170	145	200	150	10% " " " "
1849	" 9	8	200	170	180	315	120	15% " " " "
1850	" 9	8	190	200	140	260	110	20% " " " "
1851	" 9	8	160	150	125	310	120	25% " " " "
1852	" 12	8	200	200	210	215	210	6% " " " "
1853	" 12	8	190	230	200	190	250	7% " " " "
1854	" 12	8	170	200	240	180	200	8% " " " "
1855	" 12	8	200	175	190	180	150	9% " " " "

The percentages of oil are calculated from the quantity of water used.

TABLE 10.—Sand Briquette Tests with California Crude Asphalt Oils. Briquettes Weathered Wet, in the Usual Way.

Briquette No.	Date made.	Percentage of water.	Tensile Strength. 3 days.	7 days.	28 days.	3 months.	6 months.	Remarks.
1847	Feb. 9	8	170	280	210	220	320	5% Asphalt, crude 14° Baumé
1848	" 9	8	155	150	210	125	290	10% " " " "
1849	" 9	8	170	160	230	135	340	15% " " " "
1850	" 9	8	170	170	230	200	340	20% " " " "
1851	" 9	8	145	200	180	120	320	25% " " " "
1852	" 12	8	175	250	290	300	310	6% " " " "
1853	" 12	8	160	200	250	230	260	7% " " " "
1854	" 12	8	170	230	240	290	330	8% " " " "
1855	" 12	8	160	200	320	300	360	9% " " " "

The percentages of oil are calculated from the quantity of water used.

It will be noted that the tests as made, first by weathering the briquettes in water, as is customary, and also by weathering them dry, show that those which are weathered dry are apt to retrograde in tensile strength with age, while those which are weathered wet do not show this tendency. In both cases the tests show lack of uniformity. These tests were not considered to be encouraging for the use of California oil with concrete. The cement used was the Los Angeles Aqueduct brand of Monolith cement, which complies with the standard specifications for cement. Ottawa standard sand was used.

Mr. Lippincott. Six reinforced concrete siphons, 10 ft. in diameter, with pressure heads as high as 65 ft., have been built on the Los Angeles Aqueduct, and satisfactory imperviousness has been obtained by using rich cement mixtures (1:2:4), which give increased strength as well as water-tightness, instead of using water-proofing treatments which have a somewhat uncertain effect on the strength of the concrete.

Mr. Kuichling. E. KUICHLING, M. AM. SOC. C. E.—The author has presented valuable data on a very interesting subject, but he omits to indicate the trade names of the various residual oils used by him, and the places where they can be obtained. While the analyses of eight oils are given in considerable detail, it will probably be quite difficult for a practising engineer to determine whether he has in his possession an oil of the same quality as was used in the author's experiments, and hence some information as to manufacturers and uniformity of composition appears desirable.

It would also be interesting to compare the tensile strengths of the mortars mixed with all the oils mentioned, instead of only three kinds as set forth in Table 3. From this table it seems that the addition of 2½% of Oil No. 4 923 gave a much larger increase of tensile strength over plain mortar than did the same addition of Oils Nos. 4 981 and 5 053, and hence it might be inferred that the first named oil was preferable; similarly, with a 5% mixture when tested at the end of 28 days. Much variation in the crushing strength is also exhibited in Table 5, where Oil No. 4 923 seems to give the poorest results. This circumstance at once brings up the question as to the distinguishing characteristics of the oil best adapted to a particular purpose in a concrete structure. On this point further information from the author would be very acceptable.

Mr. Page. LOGAN WALLER PAGE, M. AM. SOC. C. E. (by letter).—Since the results of the writer's oil-mixed concrete investigations were recorded in the paper, additional data have been secured on the crushing strength of oil-mixed concrete at the age of 6 months and at 1 year, and are presented in Table 11.

In a general way, the results in Table 11 confirm those obtained with oil-mixed mortar specimens. The crushing strength is decreased with the addition of oil, but there is an increase with time in both air- and water-cured specimens, indicating that the oil has had no disintegrating effect during this period.

Judging from the service tests now in progress, oil-mixed concrete as a road-building material possesses no merits over those of plain concrete. It must be remembered, however, that these tests were begun before the laboratory results demonstrated the inadvisability of adding more than 10% of oil to the concrete mixture. Laboratory investigations show that oil-mixed concrete is no more resilient and

no stronger than ordinary concrete, but it possesses the property of expanding and contracting under varying conditions of moisture at a somewhat slower rate than plain concrete, because of its non-absorptive character. It is possible that, when more is learned regarding the proper method of laying an oil-concrete pavement, less disintegration will result than in plain concrete, because of the formation of fewer contraction cracks. Mr. Page.

TABLE 11.—Crushing Strength of Oil-Mixed Concrete, in Pounds per Square Inch.

Mark.	Percentage of oil.	Character of oil.	Stored in Air.		Stored in Water.	
			6 months.	1 year.	6 months.	1 year.
6c.....	0	Fluid Residual Oil, No. 4923...	980	1 090	1 955	2 170
7c.....	2½	" " " " 4923...	915	1 165	1 715	1 580
8c.....	5	" " " " 4923...	975	1 015	1 690	1 835
9c.....	10	" " " " 4923...	755	850	1 400	1 440
10c.....	15	" " " " 4923...	670	675	995	1 145
12c.....	25	" " " " 4923...	678	.....	1 080	1 120
13c.....	0	" " " " 4923...	998	1 010	1 705	1 890
14c.....	2½	" " " " 4923...	945	995	1 808	2 410
15c.....	5	" " " " 4923...	822	912	1 626	1 715
16c.....	10	" " " " 4923...	965	.....	1 406	.....

There can be no doubt, however, regarding the water-proofing possibilities of oil-mixed concrete. The writer is inclined to agree with Mr. Jorgensen, that this material, judging from its behavior in the tests thus far conducted, will be of benefit when used on the up-stream face of a concrete dam to prevent seepage. This would seem to be the logical and most economical method of water-proofing dams with oil-mixed concrete. It is possible that impervious core-walls might also be advantageously constructed of this material. The fact that at the age of 28 days there is a difference in permanent set between plain concrete and oil-mixed concrete after the application of a load of 500 lb. per sq. in., is indicative of neither the merit nor the demerit of oil-mixed concrete for use on the up-stream face in arch-dam construction. At this age it is entirely unlikely that a stress of 500 lb. per sq. in. would be applied. Tests at the age of 6 months, moreover, show the permanent set of 1:3:5 concrete, after the application of 500 lb. per sq. in., to be practically nil, whether containing oil admixtures or not, and as the modulus of elasticity is likewise the same in all the mixtures, the elastic behavior of oil-mixed concrete and plain concrete would be identical under ordinary working stresses.

Some objections offered to the use of mineral oil with cement are invalidated by the writer's experience. He has never had difficulty with the separation of the oil from the concrete mixture to the face

Mr. Page. of the forms, even when using a wetter mixture than was advisable or necessary. Again, when oil has been properly and sufficiently mixed with a stiff mortar for trowel use, it will not cause the oil to separate out on the surface. Regarding the objection that the tensile strength of concrete is weakened, nothing definite can be said, because no tensile tests have been made on concrete, nor would such an objection be of any moment. The compressive strength is somewhat weakened, which means that, if oil is used throughout the mass of the concrete—a form of construction not at all necessary—the "factor of safety," a somewhat uncertain term when applied to concrete, will be lowered.

In none of the specimens of concrete, some of them now more than a year old, can evidences of the "penetrating" character of the contained oil be discovered. Within this time the excess moisture has had plenty of opportunity to evaporate, leaving the oil free to flow.

Mr. Richardson brings to notice an interesting combination of oil with clay, this being made in the form of a slurry, to be combined with mortar or concrete for water-proofing purposes. Such a combination would have doubtful advantage over a plain oil mixture, as far as damp-proofing is concerned. In regard to Mr. Richardson's statement that these experiments arose from the work of Dr. A. S. Cushman and Mr. Albert Moyer, the writer can only say that this is entirely new to him.

In answer to Mr. Coombs' inquiry relative to the steel protective effect of oil-mixed concrete when used in a reinforced concrete pole subjected to alternate wetting and drying, and to high unit stresses, it must be confessed that the very youth of the material prevents any definite statement from being made. It might be said, however, that the mortar absorption vessels, mentioned in the paper, have remained dry on the inside for more than a year, and have absorbed only 3% of water as compared with 10.3% absorbed by the plain specimen. Steel, surrounded by oil-mixed mortar, would surely not be subjected to such severe conditions of wetting and drying as that surrounded by plain concrete. The assumption of a high working stress means the possibility of the formation of cracks through which water can enter, this being true of the oil-mixed as well as of the plain concrete. Under these conditions it is not believed that any integral water-proofing, such as oil, would aid in protecting the steel. It is generally recognized that a rich mixture of concrete, as stipulated by Mr. Coombs, is an efficient protector of steel, provided cracks can be avoided, and provided every part of the steel is thoroughly coated with concrete.

Mr. M. G. Parsons makes some potent inquiries regarding the influence of the viscosity of the oil on the concrete mixtures with which it is combined. It was found that, during freezing weather, Oil No. 4 923 was so viscous that it could hardly be poured from the barrel. In this investigation

condition it mixed with concrete with difficulty, but, after warming it to room temperature, there was no further trouble in obtaining a good uniform mixture. It would be unwise to attempt to incorporate with concrete an oil with higher specific viscosity than Oil No. 4 923. In general, it was found that the viscous oils were more effective than the fluid ones in preventing absorption, although no great difference in strength was noticed. Mr. Page.

A doubt is raised in the mind of Mr. Dunham regarding the cause of the apparent improvement in the tensile strength of 1:3 mortar when mixed with oil. It is not claimed that the improvement is due to the oil admixture, but nevertheless the improvement exists. It can hardly be due to the extra manipulation required to incorporate the oil, as the same extra working is used in making specimens for compression, but with a reversed effect on the compressive strength. As Mr. Dunham states, a 1:3 mixture should be practically impervious to water without the addition of oil, but the absorption of such a mixture is invariably high, and, furthermore, many 1:3 mixtures are very permeable. The addition of oil seems to cure these faults with certainty.

Mr. Dunham rather questions the obtaining of a public patent on oil-mixed concrete. For his information it may be stated that such a course is customary in the Department of Agriculture. The patent was obtained in order to prevent some one else from getting it and using it for commercial gain.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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TRANSACTIONS

Paper No. 1212

THE DETROIT RIVER TUNNEL.*

By Wilson Sherman Kinnear, M. Am. Soc. C. E.

With Discussion by Messrs. H. A. Carson, H. F. Dunham, Charles B. Buerger, E. M. Walker, Olaf Hoff, and Wilson Sherman Kinnear.

Historical.

That the building of some structure for the crossing of the Detroit River was not a new idea is evident from records showing that it was considered in 1867. Almost yearly since that date schemes have been presented for handling railroad traffic across this beautiful river without the use of the cumbersome car ferries. To describe only the practical plans, omitting the many fanciful structures offered by inventors who have taken out patents for methods of tunneling, of building submerged tubes and new styles of truss bridges, would be a task to appal the most ardent enthusiast after only a brief examination of the many drawings, descriptive and otherwise, and the numerous files and other archives on this subject. Among the many plans seriously considered, the most noteworthy are: The tunnel started in 1872, and afterward abandoned, by the Michigan Central Railroad and the Great Western Railway (now a part of the Grand Trunk system); a draw-bridge in 1873; a high bridge in 1879; the winter bridge in 1883; a high bridge again in 1896; a high bridge in 1904; and the present tunnel plan, developed since the summer of 1904.

By reference to a map it will be readily observed that the Detroit River is one of the links in the chain of Great Lakes and the system of

* Presented at the meeting of October 4th, 1911.

connecting waters. It extends from Lake St. Clair to Lake Erie, a distance of some 28 miles. Its general direction is southwest, and, at Detroit, it is possible to perform the apparent paradox of going south to Canada. The river is about ½ mile wide at Detroit, and its depth is from 18 to 48 ft.; its current varies as the wind blows up or down stream.

The commerce of the Great Lakes, consisting of great quantities of iron ore from Lake Superior and wheat from Lake Michigan points to their destinations at Buffalo and other Lake Erie ports, and coal from Lake Erie and Lake Superior ports, must all pass through this magnificent river, often termed the "Straits of Detroit." The commerce of the Lakes passing Detroit is of a magnitude rarely known or appreciated. It exceeds in tonnage that of Boston, and approaches very closely the combined foreign and coast commerce of New York City. That passing the site of the tunnel in 1906 was more than 50 000 000 tons, as shown by Table 1.

TABLE 1.—Vessels Passing Detroit.

Year.	No. (Estimated.)	Number of tons registered. (Estimated.)	Value of cargoes.
1879	18 794	12 000 000	
1902	33 000	39 328 098	$440 834 640
1903	33 113	37 453 796	471 917 880
1904	29 472	33 049 984	453 598 656
1905	35 599	45 912 622	522 888 751
1906	35 128	50 678 897	662 971 053
1907	34 149	53 959 769	697 311 302
1908	27 883	40 628 850	614 425 480

The following will serve to show the volume of carload traffic handled by the railroads across the river at Detroit yearly from 1875 to 1878, inclusive, and from 1897 to 1909. These figures have not been selected, but are published records. The marked yearly increase should prove interesting and instructive.

Year.	No. of Cars.	Year.	No. of Cars.
1875	173 708	1897	507 894
1876	192 107	1898	575 894
1877	179 716	1899	624 802
1878	210 490	1900	621 073
		1901	650 117
		1902	590 486
		1909	735 753

The railroads handling traffic across the Detroit River are equipped with large, self-propelling car ferries, operated not only at a great expense, but at a decided disadvantage during the winter, when the river is filled with ice. These boats are of various designs, carrying either two or three tracks. Each track of the largest boats will hold from six to eight freight cars, or three or four sleeping cars or coaches. Experience in operating them, covering a period of years and including all designs, has shown them to average about twelve cars per trip. This average is based on all classes of cars. It will thus be seen that through trains have to be cut into many sections and re-assembled on the opposite side of the river, requiring a large number of switching movements. In crossing the river and passing through the terminal yards contiguous thereto, freight trains consume from 3 to 8 hours within the approximate 5-mile limit of such operations, and a large percentage of this time is direct delay incident to transfer by ferry; the interruption to passenger trains is not nearly so great, due entirely to the character of the service.

Up to the time of the opening of the tunnel, when the Michigan Central Railroad abandoned its ferries, ten car ferries were used for the transfer of the railroad traffic at Detroit; nine were in constant service and, during the season of 1909, ferried 735 753 cars. As before mentioned, the boats are of various sizes and designs, the largest being 351 ft. long and carrying 24 freight cars. The majority are side-wheelers, but those of the most modern design have propellers fore and aft. The most successful iceboat has side-wheels and a single propeller. Aside from interference by the through commerce of the Great Lakes, there is no great difficulty in operating the boats during the summer; but, from December 1st to April 1st, the ice is a very serious menace to the safe and expeditious movement of these vessels. It is not often that they are stopped by thick ice, the interruption being caused in the main by slush and anchor ice near the shores. The accumulation of anchor ice in and near the slips frequently reaches to the bottom of the river, thus making a landing at times a question of hours.

The crossing of the different routes by the ferries causes the ice broken in each channel to drift downward, filling the one below, and thus establishing as the easiest crossing the one farthest up stream.

During the winter of 1887-88 the most powerful ferry then constructed, and now in operation, was fast in the ice for four days and

unfit for service for a month. The river was blocked for two weeks, and traffic had to be diverted *via* Toledo to the East, and also by way of Port Huron, where the channel happened to be open. The winter of 1903-04 was unusually severe, and one boat was stuck in the ice for several days. This was in spite of the fact that the largest and most powerful ferry had just been put in service. During every winter a certain amount of traffic is diverted from the Detroit crossing to railroads to the north and south, and this diversion becomes more or less permanent both for passenger and freight traffic. In 1888 some elaborate figures were prepared showing a marked decrease in the percentage of through passenger business between the summer and winter. Between Buffalo and Chicago, the Lake Shore and Michigan Central Railroads have always been great rivals for the passenger business, and for the year ending April 30th, 1888, during the summer, the Lake Shore carried 54.61% of the passengers; during the winter, however, it carried 61.58 per cent. In other words, the Michigan Central's percentage decreased from 45.39 to 38.42% of all the through passengers carried by the two roads. The value of the passenger business thus lost cannot well be estimated, and once diverted may not return.

The loss to the Michigan Central, or to any railroad, by delay to freight, is equally hard to estimate. The fluctuation in earnings forms the best basis, and direct comparison can be made with operating costs. The results have shown enormous losses directly chargeable to the river crossing, and it is but fair to assume that a large percentage of the business lost by diversion is not recovered.

From this brief discussion of the commerce through, and the traffic across, the Detroit River, the reasons for long-continued study of the river-crossing problem by the railroads can be more readily understood and appreciated. Four very determined efforts were made to secure necessary Congressional legislation for building a bridge across the river, the first in 1873 and the last in 1896.

In 1873 a board of United States Engineers, under the direction of the War Department, spent the summer and fall collecting data as to commerce and river conditions at Detroit, making a report on December 24th,* with the following conclusions:

* Report of the Chief of Engrs., U. S. A., 1874, Pt 1, p. 602.

"1st. That a bridge giving a clear headway of 150 feet and clear spans of 400 feet, would not seriously injure navigation, * * *.

"2d. That no bridge giving passage to vessels by draws alone, with draw-spans at present practicable, can be permitted without serious injury to navigation.

"3d. That a bridge giving a clear opening of 700 feet from April 1 to December 1, with two draw-openings 100 feet in the clear, and with the permanent foundations of its movable piers 18 feet below lowest stage of water, will not be a serious obstacle to navigation.

"4th. * * * the board deem the crossing of the river by tunnels the only unobjectionable method * * *."

From this it will be seen that even the slight obstructions to navigation offered by the type of bridge considered did not meet with the approval of the Board of Engineers. The recommendation of the tunnel was voluntary.

In 1879 another effort was made to secure a charter for a bridge, with the result that a second Board was appointed, composed, as before, of U. S. Engineers, one member being the late Gen. O. M. Poe, M. Am. Soc. C. E. This Board also favored a tunnel, but was of the opinion that with skillful navigators there would be no material difficulty in passing through clear openings of 300 or 400 ft. at any time when it would be safe to navigate the river.

The fixed spans of the proposed bridge were to have clear openings of 450 ft., and the head-room was established at only 60 ft. One member of the Board would not concur as to the draw-bridge, and Congress took no action.

Again, in 1889, a board was appointed to investigate the crossing of the river by a bridge. This time the bridge considered was of novel construction. It was to be built in eleven spans, all except three of which would be removed in the summer, and the three central piers, supporting the four spans and covering a distance of 1 000 ft. in the center of the river, were to be removed to a depth of 40 ft., so that no vessel would ever come in contact with them. Fig. 1 is a profile of this novel scheme. The bridge proposed was for winter use only, hence its name, "The Winter Bridge." A strong effort was made to induce the Board to report favorably on this scheme, and a large amount of testimony was taken, which included testimony regarding a high bridge plan submitted by a third party. The Board* concluded as follows:

* Report of the Chief of Engrs., U. S. A., 1890, Pt. 4, p. 3461.

"First. In their opinion it is feasible to build a bridge across Detroit River at Detroit, Mich.

"Second. That of the projects for a bridge submitted to them, that for a high bridge proposed by Mr. Lindenthal is the least objectionable and the most conservative of the shipping interests, and therefore the best plan.

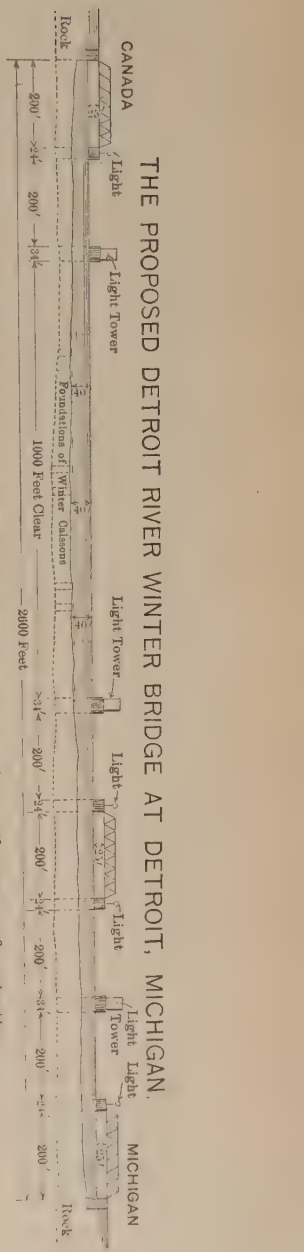
"Third. That project No. 2, for a winter bridge is the most practicable because it will accomplish the result sought at less cost, though justly subject to greater objection upon the part of navigators * * *."

It will be seen that for a third time the railroad failed to receive favorable consideration at the hands of the U. S. Engineers.

In 1896 another attempt was made to secure the necessary legislation for a high bridge. Under plans submitted at this time, the piers in the river were to be 1 130 ft. apart, but, as before, sufficient opposition was introduced to prevent the necessary action being taken. The opposition did not develop until the matter came before the Senate.

In 1904 the Grand Trunk Railway joined with the Michigan Central Railroad and employed the late George S. Morison, Past-President, Am. Soc. C. E., to select a site for a high-level bridge, which would be the best for all the railroads interested in the Detroit River crossing. The report made by Mr. Morison's successor had just been submitted, when the development of electric traction for heavy railroad traffic by the New York Central and Hudson River Railroad Company diverted somewhat the line of study, and assured the feasibility of the application of electric power to main-line traffic on heavy grades, thus eliminating one of the greatest objections to a tunnel,

THE PROPOSED DETROIT RIVER WINTER BRIDGE AT DETROIT, MICHIGAN.



This represents the bridge as proposed to be opened and lighted during the season of navigation. It was proposed to have one main opening of 1000 ft. and four of 200 ft. each; Total clear opening=1800 feet

FIG. 1.

Canadian end was located entirely on property owned by the Michigan Central, but it forced the rebuilding of a terminal yard, as explained elsewhere.

In order to determine the nature of the material through which the tunnel was to pass, it was decided to take extensive wash borings throughout the entire length of the tunnel construction. Due to a possibility of the use of the shield method, it was not thought desirable to make borings in the exact line of the tunnel in the river section. As a result, two lines of holes were bored, 100 ft. down stream and 100 ft. up stream from the center line between the two tunnels; on these lines the borings were 100 ft. apart, and were carried to bed-rock. By reason of the uniformity of the material, this method gave sufficiently accurate determinations between the lines where the samples were taken. The work was done with the ordinary churn drill, consisting of a hollow 2½-in. casing, enclosing a $1\frac{5}{16}$-in. drill-bar, through which the water was forced to the bottom of the rod and returned through a perforated bit. As the water was forced upward from the bottom of the hole, it carried with it small particles of clay. The samples were collected at the top of the casing and preserved. The churning of the drill-bar cut the core to a large extent, and it was often impossible to determine accurately the exact nature of the material encountered. A more accurate determination was often made by withdrawing the drill-bar, taking off the bit and returning the bar, without closing the opening at the lower end. The bar was then forced and driven into the material until several feet of clay had accumulated. It was then withdrawn, water pressure was applied to the part which contained the clay, and the core was forced out. This core gave an accurate determination of the material, and sections were preserved in air-tight bottles for inspection. Throughout the limits of the tunnel the material was found to be a coarse, blue clay, of varying consistency, with occasional seams of sand and gravel, and scattered boulders. In addition to the borings, four test-pits, two on each side of the river, were sunk, and served to verify in every particular the character of the material as disclosed by the borings.

The river borings cost about 65 cents per lin. ft. More substantial floating equipment would undoubtedly have materially reduced this cost, although delays and interruptions are unavoidable in work of this description.

Alignment.—As the tunnel lies wholly within the limits of the right of way of the Michigan Central Railroad, it was thought advisable to make a complete new survey of all tracks and buildings, both in Detroit and Windsor, which might be affected by the work of construction, as previously mentioned. For convenience, the gauge line of the east-bound main passenger track was taken as a base line on both sides of the river, and to this all re-surveys were referred. The stationing was assumed so as to bring the zero mark about 1 200 ft. west of the Detroit city limits, and the numbering was carried continuously across the river on the preliminary triangulation of the Windsor right of way.

For the preliminary triangulation across the river, a monument, formed by a rail splice-bar, was located in the Detroit yards, near the present site of the Detroit shaft, and for its stationing was tied into the re-survey base. On the Windsor side a monument was located near the river on the line of the gauge of the south rail of the east-bound main track, on the tangent east from London Street. To triangulate for the distance between these two points, a base was established on the Detroit side measuring 1 963.82 ft. A base was also established on the Windsor side measuring 1 607.40 ft. Only ordinary care was used in measuring these bases, as the survey was being platted to a scale of 50 ft. to 1 in. Each base was measured twice, checking within 0.05 ft. The angles were read with five repetitions and the means taken, which closed to 15 seconds. The two triangles gave values for the distances across the river varying by 0.37, and the mean was taken, which was close enough for platting the map on the scale used. The station of the Windsor monument was thus determined, and from this point the line was carried eastward along the tangent of the east-bound main track.

Elevations were taken on the top of the rail used as the re-survey base, to which were referred the levels for cross-sectioning at each station, and the topography. The datum used was that of the United States Government, being the elevation of mean tide at Sandy Hook. The surface of the Detroit River at the site of the tunnel, referred to this datum, has an average elevation of 574.2 ft.

Preliminary soundings to determine the depth of water in the river were taken in advance of the definite location of the tunnel line. These were supplemented by more accurate data, thus giving a correct

cross-section of the bed of the river. The preliminary soundings were taken with an ordinary lead and line from a tug, the locations being observed by two transits set up at the ends of the preliminary base line on the Windsor side. The more accurate soundings were taken with a special machine, which consisted of a reel on which was wound a sounding line of steel piano wire. The plummet was of iron, weighing about 90 lb., of pointed oval form, with a wooden vane to keep it pointed up stream so as to offer little resistance to the current. The machine was mounted on a flatboat formed by two wooden pontoons, the wire passing down through the opening between them. The locations of these soundings were taken in the same manner as before. The soundings indicated a maximum depth of 48 ft. and a minimum of 18 ft.

There are two channels in the river in front of Detroit, one known as the Canadian channel, near the Canadian shore, and the other the central channel, located, as indicated by its name, near the center of the river. Observations covering a long period of years indicate that 90% of the commerce of the river passes through the Canadian channel.

Measurements to determine the current velocities along the tunnel line were taken in August and September, 1905, and showed a maximum velocity of 3.4 ft. per sec., or 2.32 miles per hour. The velocities recorded were sometimes greatly increased by strong southwest winds, when the water in Lake Erie is driven east, thus lowering its level at the mouth of the Detroit River and increasing the difference in level between Lake Erie and Lake St. Clair, thereby increasing the velocity of the current in the river, so that in all probability the maximum velocity is not far from 3.5 miles per hour; and one or more of the subaqueous sections was lowered when this maximum obtained. The extreme variation in the water level of the Detroit River is as much as 3 ft. above or below the mean elevation, 574.2 ft. The level has changed as much as 3 ft. in as many hours. As the direction of the wind has such a decided effect on the current in this part of the river, it is unfortunate that the most favorable weather prevailed when this work was in progress, hence there were no particularly interesting disclosures as to varying conditions.

The final survey consisted in the establishment of permanent monuments and bench-marks to be used as reference points for setting line, grade, and station points to control construction. Monuments were set

on the approach tangents at the summits, at the points of intersection of the approach tangents with the river tangent, and at several convenient intermediate points. These generally consisted of pieces of scrap rail, about 4 ft. long, set in a 9-in. post-hole, with the top of the rail 1 or 2 in. above ground. The monuments were carefully tamped and allowed to settle for a day or two before marking. The river tangent was produced both ways, beyond the points of intersection, far enough to reach high ground not likely to be disturbed, and monuments were set. These consisted of old 15-ft. rail switch-points, driven point down about 4 ft. into the bottom of a 10-ft. hole which was excavated so as to give a bottom diameter of about 3 ft., tapering to 2 ft. at the top, and the hole was filled with concrete around the rail. Witness monuments, consisting generally of scrap rail, were set so as to be readily accessible for checking the tangent monuments and, at the same time, not be likely to be disturbed. The east-bound tunnel had its approach tangents 21.5 ft. and its river tangent 26.4 ft. south of, and parallel to, those of the west-bound tunnel; no monuments were set especially for the east-bound tunnel, all lines for the latter being offset from the monuments set for the west-bound tunnel.

The final triangulation for the distance across the river, along the monumented west-bound tangent, was made from two base lines, one on each side of the river, the length of the Detroit base being about 942 ft., and that of the Windsor base, about 1 545 ft. In measuring these bases the tape was supported on stakes, to bring it to a definite profile, the temperature correction was made, and a spring balance was used to indicate the tension, so that the proper comparison could be made with the U. S. Government standard. The mean values of a series of measurements on each line were taken, the extreme variation being 0.013 ft. for the Windsor base and 0.004 ft. for the Detroit base.

For the angles of the triangles, a "Berger" engineer's transit, graduated to read to 30″, was used. Special targets were devised for making long sights across the river. These consisted of pieces of board, about 12 in. wide and 20 in. high, painted white, on which were ruled two lines in the form of an **X**, having a slope with the vertical of about 22 degrees. The space on each side of the **X** was painted black, leaving two white isosceles triangles meeting in a point. In making sights across the river, the vertical cross-wires, as projected on the target, covered a space about $\frac{3}{4}$ in. wide, but by using this form

of target it was possible to bisect the white triangles accurately, as the steep slope of the X made the apparent position of the points of the four triangles, appearing on either side of the vertical wire, sensitive indicators of error in sighting, whatever the apparent thickness of the wire. In reading the angles at each station, fifteen repetitions were added on the limb, reading from right to left, then fifteen turning left to right, and then two more sets of fifteen repetitions with the telescope reversed. Most of the readings were taken on cloudy days with a clear atmosphere, although during part of the time there was bright sunshine. This did not interfere materially, provided the sights were across open spaces and not alongside vertical surfaces which would reflect the heat and cause horizontal refraction.

The angles on each triangle were adjusted by distributing the error of closure among them. The angles for the triangle on the Windsor base line closed to 2.55″ less than 180°, and those for the triangle on the Detroit base line closed to 2.08″ less than 180 degrees. The distance across the river, along the west-bound, monumented tangent, computed from the triangle using the Windsor base line, had a value of 2 717.149 ft. The value given by the triangle on the Detroit base for this distance was 2 717.171 ft., making a difference between the two triangles of 0.022 ft., which was considered a very close check. These values were adjusted by giving a weight of fifteen to the length given by the triangle on the Windsor base line, and a weight of nine on the other triangle, being practically the ratio of the lengths of the base lines. Measurements through the completed tunnel check the triangled distances across the river almost exactly.

To obtain the exact values of the intersection angles between the tangents, the angles were read as in the triangulation. The targets for the sights across the river were similar to those described, but about twice as large. At the point of intersection of the river tangent with the Detroit approach tangent, a special tripod, about 10 ft. high, was erected over the monument, so that sights could be taken over box-cars which might be in the yards. This was of double construction, having one tripod inside and independent of the other, the inner one supporting the instrument and the outer one carrying a platform on which a man could stand without disturbing the instrument.

Stationing was adopted on the west-bound river tangents to correspond with those on which the borings in the river bottom were made.

This was also adopted later for the east-bound tangent, so that stations of the same number are opposite each other on the river tangents, allowing differences to accumulate around the curves to the approach tangents in both directions.

For transferring levels across the river, a special target, of heavy cardboard, 4½ by 7½ in., was ruled and painted like those used on the triangulation work. The target, however, was placed on the regular brass target, with the altitudes of the white triangles horizontal instead of vertical, and the long dimension of the card horizontal. The center of the target was set 2 in. above the zero of the vernier, and the card was notched sufficiently to permit the reading of the vernier, and to clear the head of the clamp-screw. In taking the levels, two 22-in. wye-levels were used simultaneously, one on each side of the river. Each was set up as near the dock front as practicable, and at a distance of about 200 ft. from the turning point, which was established on the dock, the distance between the turning points on the two sides of the river being about 2 400 ft. Fifteen readings were taken from each instrument on the near turning point and on that across the river alternately, white flags being used in signaling. The instruments were then exchanged, special care being taken not to disturb their adjustment, and two additional sets of fifteen readings were taken. Then the rods and rodmen were interchanged for a third set, and, lastly, the instruments were returned to their original stations for the fourth set. The mean of the eight sets of readings was computed for the difference of elevation of the turning point. By adopting this programme, all corrections for curvature of the earth's surface, refraction, inequality of back-sights and fore-sights, lack of adjustment of the instruments, and differences in the rods, were taken care of automatically. It was found impossible to work when the sun was shining, owing to variable refraction and heat waves, but with fine weather conditions, such as a cloudy sky and clear air, good results were obtained. The notes for any set of fifteen readings show no greater extreme variation than 0.08 ft., and three sets of twenty readings show an extreme variation of 0.03 ft. The difference in elevation between the turning points on the two sides of the river, as found, was fairly close; when checked by levels through the completed tunnel the error was 0.01 ft.

Line and grade points for the work in the headings were transferred from the surface through the various shafts. Monuments which

were thought to be substantial were set in the headings, but it was found that all the points shifted more or less, owing to the unstable character of the clay, and had to be checked from the surface about once a week. Line points for the curves were referred to tangents, chosen so as to fall within the drifts. Offsets from these tangents at intervals of 5 ft. were computed to give the true line.

Organization.—In July, 1905, the Board of Directors of the Michigan Central Railroad Company appointed an Advisory Board of Engineers to have charge of the construction and electrification of the tunnel, consisting of the following members: William J. Wilgus, M. Am. Soc. C. E., Chairman; Howard A. Carson, M. Am. Soc. C. E., and the writer, as Chief Engineer. The local staff, immediately appointed, was as follows: Benjamin Douglas, M. Am. Soc. C. E., Tunnel Engineer; J. C. Mock, Electrical Engineer; A. C. Everham, Assoc. M. Am. Soc. C. E., Terminal Engineer and, later, Assistant Tunnel Engineer; and J. L. Holst, Designing Engineer. At about this time all the previously organized bridge and tunnel companies were consolidated under the name of the "Detroit River Tunnel Company," by which company the tunnel was built.

In advance of construction work, a complete testing laboratory was designed and built, on the water front in Detroit, on property owned by the Michigan Central Company. The main building is 22 by 48 ft., two stories high, and of fire-proof construction, with plaster walls, inside and out. The inspection of material was a very important feature of the tunnel work, and particular attention was given to cement. All cement used in the work was sampled at the mills, sealed in bins, tested at the laboratory, and shipped by the inspector from the mills in cars sealed by him. The seals were inspected at the destination and broken by a representative of the Tunnel Company before the contractor was permitted to use the cement. In this manner the contractor was relieved of all responsibility of inspection, and the cement was delivered to him as accepted. The cost of this testing laboratory, including equipment and a 300 000-lb., Olsen, screw, testing machine, was about $20 000.

For the convenience of the engineers and inspectors, two field offices were built, one in Detroit and one in Windsor. These buildings contained lockers, shower baths, storerooms, dining-room, and kitchen. Meals were served four times a day for all employees of the Tunnel

Company. In the Detroit field office, a complete photographic outfit was installed, and a photographer was employed to take progress pictures of the work.

Plans, Contract, Etc.

The work of preparing plans was started in August, 1905, and by February, 1906, the contract drawings and specifications were handed to contractors with the request that proposals be submitted by March 26th, 1906. To aid in an analysis of the proposals, the entire work was divided into sections, as follows: westerly open cut; westerly approach tunnel; subaqueous section; easterly approach tunnel; easterly open cut; Detroit shaft; and Windsor shaft. Contractors were given the option of bidding on any or all sections.

The preliminary plans covered, not only approved designs for all the sections, but four alternative methods for constructing the subaqueous section. The contractors were given the option of selecting any one of the four methods for the subaqueous work, or submitting entirely new designs, or modifications of those suggested, restricted only to a compliance with certain conditions regulating stability, clearances, workmanship, etc.

Three of the four methods suggested for the subaqueous section were entirely new, as applied to work of this magnitude, and the four alternative suggestions may be briefly described as follows:

Design A.—In its inception, as presented by Mr. Wilgus, this design contemplated the dredging of a trench across the river, on the line of the tunnel, of proper width, and to an established grade; the sinking and connecting of cores or forms to line and grade; the surrounding of these cores or forms with tremie- or bucket-deposited concrete; the unwatering of the interior of the cores or forms, and building in the opening thus provided a lining of reinforced concrete of dimensions sufficient to guarantee stability, continuity of strength, and water-tightness.

This method, although distinctly novel, presented several advantages, such as probable economy in construction, adaptability to higher grade line, and freedom from high air pressures—sufficient in themselves to demand careful study of the basic principles.

Design B.—This design, as presented by Mr. Carson, involved the dredging of a trench and the lowering to correct position therein of completed sections of a reinforced concrete tunnel, to meet the same

requirements as those mentioned under Design *A;* the sections to be built in dry dock, towed to position, sunk, and joined under water. A sewer tunnel of comparatively small dimensions was built by Mr. Carson several years ago along these lines.

Design C.—This method was in the main a modification of Design *A,* and contemplated the use of steel forms.

Design D.—A shield-driven tunnel was specified under this heading, using cast-iron segments for preliminary protection, to be followed by a lining of concrete. In order to preserve the established gradients for the approaches and the economical location of open cuts, had this method been used, the covering over the shield operations under the major portion of the river would have been so light, without expensive blanketing, as to render the work extremely hazardous, if not entirely impracticable. The lowering of the tunnel grade line in its entirety, to avoid this marked element of danger, would have carried the shield into solid rock for at least 1 200 ft. in the center of the river.

The wording of the invitations to contractors, supplemented by the suggested designs, resulted in a number of bids and the submission of four independent designs. A very careful canvass of the entire situation by the Advisory Board resulted in awarding the contract for all the work to the Butler Brothers-Hoff Company.

The design for the subaqueous section submitted by this firm, was a modification of Design *A,* embodying some of the elements of Design *C,* accompanied by a large amount of detail covering the methods to be used in the prosecution of the work.

A lump-sum price was submitted for each section of the work, in addition to a schedule of unit prices covering various items. This schedule formed a part of the contract, to be used for the adjustment of changes in contract drawings as applicable to increased or decreased quantities.

The contract was awarded on August 1st, 1906. The work was started on October 1st of the same year, and completed on July 1st, 1910.

Tunnel.

The tunnel work was naturally divided into three sections: The subaqueous, the westerly approach, and the easterly approach. The subaqueous section is 2 668 ft. long; the westerly, or Detroit, approach is 3 669 ft. long, including an open cut of 1 510.5 ft.; and the easterly

PLATE XXXIV.
TRANS. AM. SOC. CIV. ENGRS.
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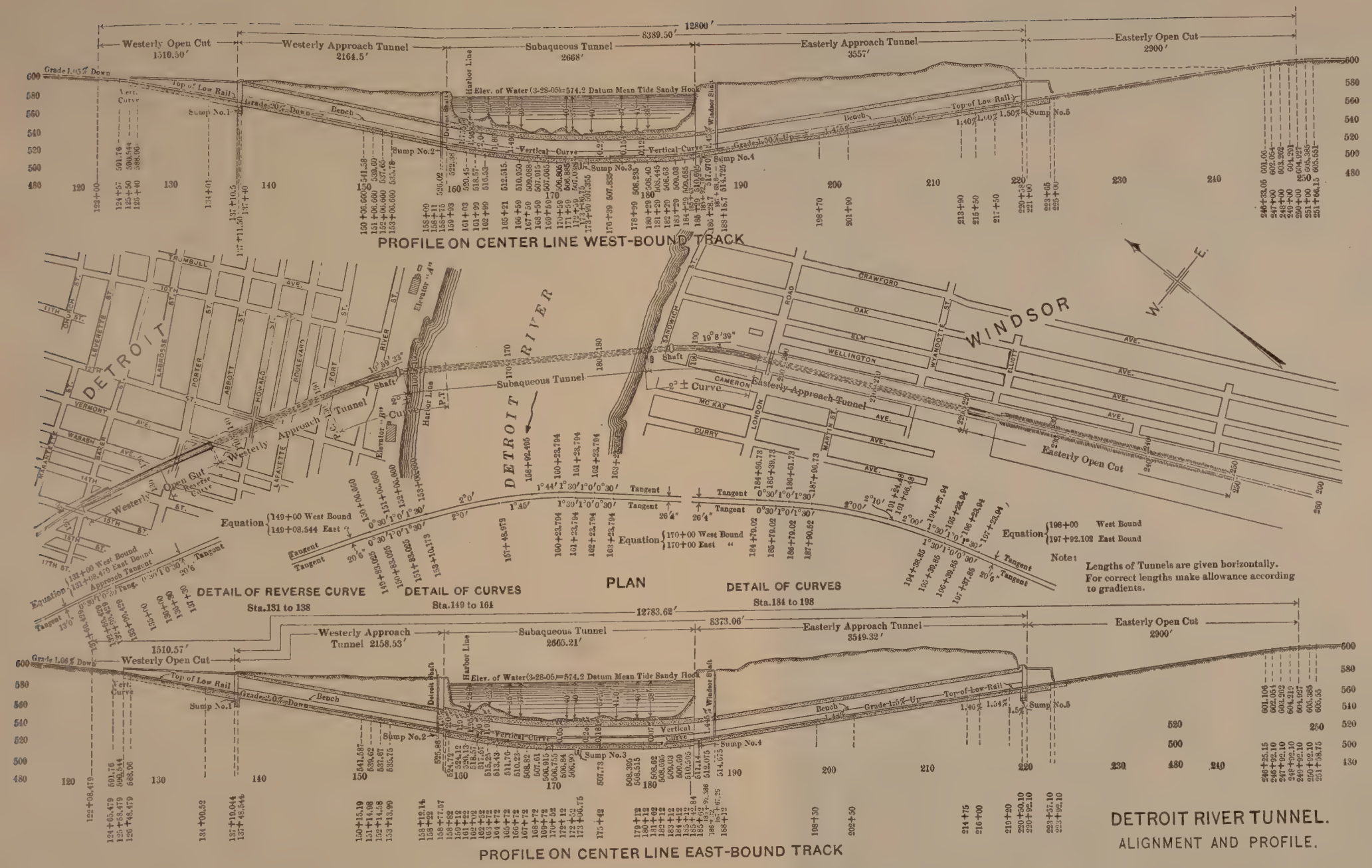
KINNEAR ON
THE DETROIT RIVER TUNNEL.



approach, in Windsor, is 6 449.2 ft. long, including an open cut of 2 900 ft.

The general location of the tunnel and approaches is shown on Fig. 2. The alignment and profiles are shown on Plate XXXIV. The tunnel consists of two single-track tubes, with a maximum gradient of 1.5% on the Windsor approach and 2% on the Detroit approach, compensated for curvature, and a maximum curvature of 2 degrees. The gradient for the Windsor approach was made the lower, favoring the heavier east-bound tonnage.

As previously stated, the entire structure is built through blue clay, the character of which varies considerably along different sections of the work. In Detroit it is fairly stiff, except for a distance of about 450 ft. back from the river, where it was soft enough to require the use of compressed air in the headings. There was an especially soft spot in this latter section adjoining an old test-pit, where the ground had become water-soaked. In Windsor the clay is more or less soft throughout, and when exposed in headings had such a tendency to flow that it was found necessary to use compressed air for nearly all tunnel work on that side of the river. Under the river the clay underlying the tunnel is very hard near the Windsor side, comparatively soft near the middle, and quite firm again approaching the Detroit shore.

Some tests made by the Tunnel Company in the easterly and westerly open cuts and in the headings in Windsor, shortly after excavation was started, with a view of determining the sustaining power of the clay, showed that a load of from 5 000 to 12 000 lb. per sq. ft. could safely be imposed at any point in the approaches without causing marked settlement. No reliable tests were made of the clay in the river-bed, in view of the results obtained in Windsor, where the original borings had shown it to be quite as soft as that under the river. Furthermore, it was demonstrated that any test made in from 60 to 80 ft. of water, where all the conditions could not be observed, would not be of much practical value.

The approach tunnels are built of plain concrete, without reinforcement, except for some longitudinal rods in the inverts, short transverse rods under the ties, and, for a short distance in Detroit, some special reinforcement where local conditions demanded it. The retaining walls in the open cuts are of the gravity type, with steel rails placed longitudinally in the footings and vertically in the back of the walls.

The subaqueous section of the tunnel is of unique construction, involving new methods of work, and will be described in detail later.

Westerly Open Cut.

The westerly open cut is along the northerly right-of-way line of the Michigan Central Railroad, in Detroit, extending from Fifteenth Street to Vermont Avenue. On account of the limited width of the right of way at this point and the consequent need for preserving as much of it as possible for yard purposes, the cut is enclosed by retaining walls. A cross-section of these walls, as built near the portal of the tunnel, is shown on Plate XXXV. The execution of this work involved the removal of the three most northerly tracks, including the west-bound passenger track, leaving the east-bound passenger track, which was operated during the work as a single track, and three through tracks south of the single main track. The east-bound track, on account of its proximity to the cut, was supported on piles east of Baker Street.

Excavation.—The excavation was made entirely by steam shovel, which necessarily worked in rather cramped quarters on account of the limited width of the cut, and was somewhat handicapped by having to pass close to several abutments and piers and over quite a number of city water mains. Further interruptions were caused by an infrequent train service, which was necessarily poor on account of the limited yard room. In spite of the many drawbacks, however, the steam-shovel work proved to be economical. The total excavation for this cut was about 38 000 cu. yd.

Concrete.—To secure a structure as nearly monolithic as possible, the concrete in the walls was placed in sections 48 or 50 ft. long, and in one run from the top of the footing to the under side of the coping. The backs of the walls were water-proofed by the application of one layer of coal-tar pitch down to the weep-holes, which are spaced at about 15-ft. centers along the walls. Where the walls are high and the pressure rather heavy on the underlying material, inverts were constructed, as shown on the cross-section, extending from wall to wall. Where the invert was not used, struts, 2 ft. square, were placed at intervals of about 15 ft., thus providing at every point for holding the walls apart.

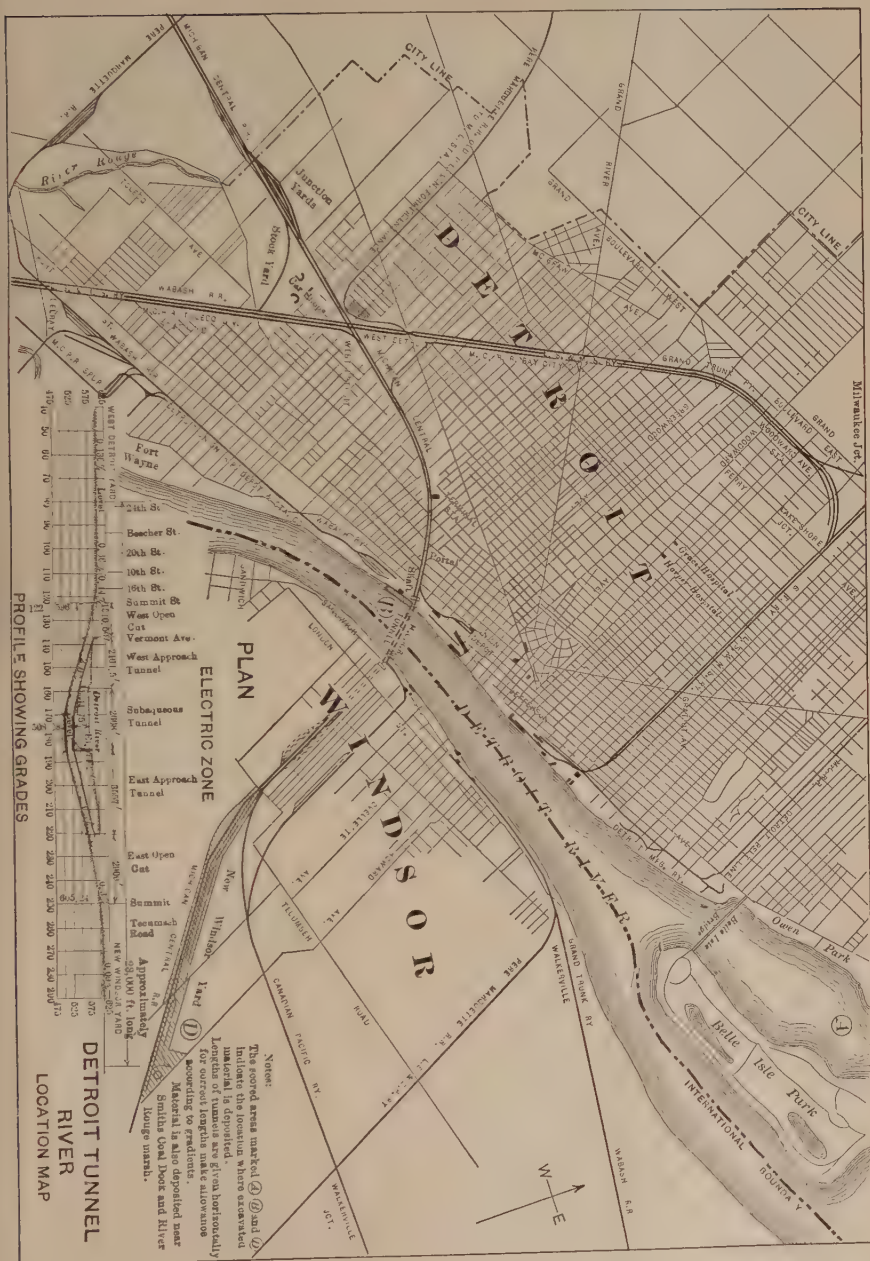

DETROIT RIVER TUNNEL
LOCATION MAP
PROFILE SHOWING GRADES
PLAN
ELECTRIC ZONE
Notes:
The scored areas marked A B and D indicate the location where excavated material is deposited.
Lengths of tunnels are given horizontally for correct longitudinal measurements according to grades.
Material is also deposited near Smiths Coal Dock and River Rouge marsh.
Fort Wayne
Junction Yards
Stock Yard
Belle Isle Park
Owen Park
Milwaukee Jct.
Grace Hospital
Harper Hospital
Fort Wayne
24th St
Beecher St.
20th St.
10th St.
16th St.
Summit St
West Open Cut
Vermont Ave.
West Approach Tunnel
Subaqueous Tunnel
East Approach Tunnel
East Open Cut
Summit
Tecumseh Road
New Windsor Yard
Approximately 23,000 ft. long
WEST DETROIT YARD
NEW WINDSOR YARD
Detroit River
Level
CITY LINE
INTERNATIONAL BOUNDARY
WABASH R.R.
GRAND TRUNK RY
WALKERVILLE
CANADIAN PACIFIC RY.
TECUMSEH ROAD
PERE MARQUETTE R.R.
MICHIGAN CENTRAL R.R.
River Rouge
GRAND RIVER AVE.
GRAND BOULEVARD WEST
GRAND BOULEVARD EAST
WOODWARD AVE.
FERRY
LAKE SHORE JCT.
JANIWICH
EDBURGH
Point
Ports
W
E

Fig. 2.

Within the limits of the cut, the track centers are reduced from 20 ft. 6 in. at the portal, which is the distance between the tracks on the tangent in the approach tunnels, to 13 ft. 0 in., the standard distance between the main-line tracks on the Michigan Central Railroad. This was done within the limits of the cut, so as to retain as much of the yard room as possible for future use. The change in line was made entirely in the east-bound track, and the south wall was built on a reverse curve, leaving the north wall on a tangent.

The walls are of three grades of concrete: A 1:4:7½ mixture in the footings; a 1:3:6 mixture in the body of the wall; and a 1:2:4 mixture in the coping. The specifications required a mortar face, but it was found that better results were obtained by carefully working the concrete at the face of the wall.

The fence on the top of each wall is composed of three lines of wrought-iron pipe, supported at intervals of 6 ft. by vertical wrought-iron posts, 2 in. in diameter. The diameter of the upper line of pipe is 2 in., that of the two lower lines is 1½ in., and all are parallel with the coping. Over every joint in the retaining wall a length of pipe slips into a sleeve on one of the posts, so that the expansion and contraction of the pipes will not mar the alignment of the fence.

Drainage.—To prevent the collection of water behind the walls, in addition to the weep-holes, blind drains extend from the top of the wall to the weep-holes, and there is a porous back-filling of cinders or gravel. The method of taking care of the water which will naturally drain to the open cuts and the tunnel will be described later. As much surface-water as possible is caught in catch-basins and conducted directly to a sewer before it reaches the cut.

Design of the Approach Tunnels.

Inasmuch as the approach tunnels, as built, have two distinctly different cross-sections, it is appropriate that conditions giving rise to such difference in design should be explained at the outset. Cross-sections of the approach tunnels are shown on Plates XXXVI and XXXVII. Types B, C, D, E, and F, represent practically the original designs, made before the contract for the work was awarded, when it was expected that the tunneling for the approaches throughout would be made in several headings or drifts, which would include the walls,

PLATE XXXV.
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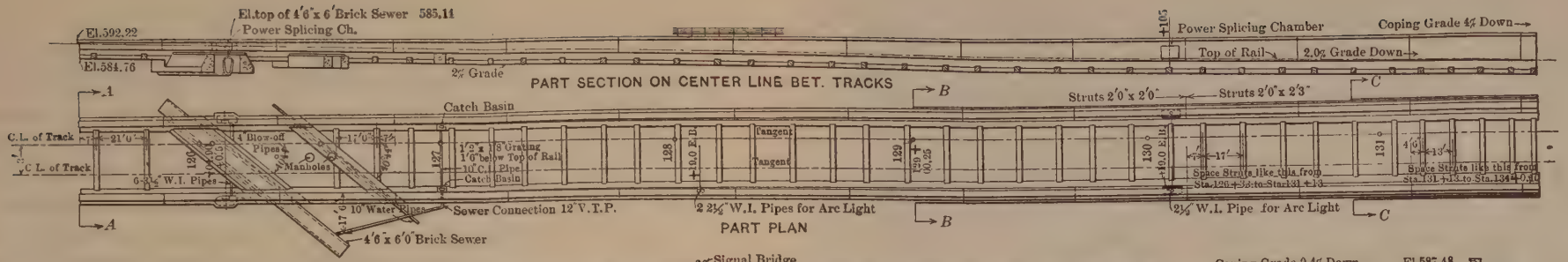
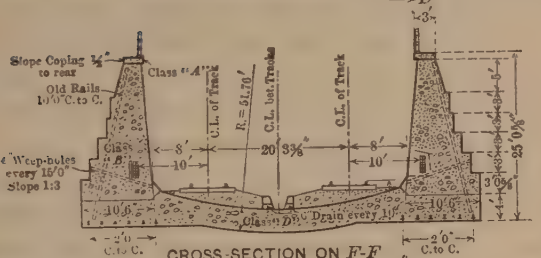
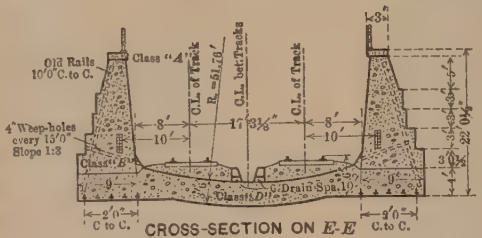
THE DETROIT RIVER TUNNEL.







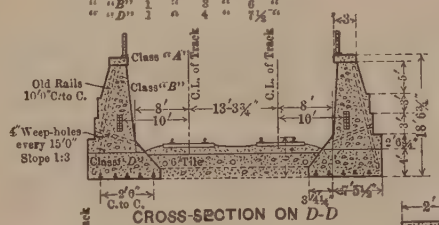
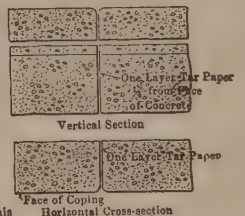
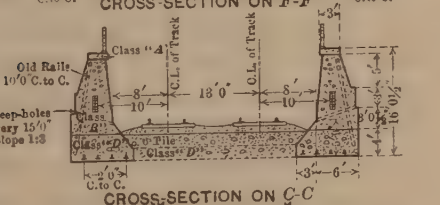
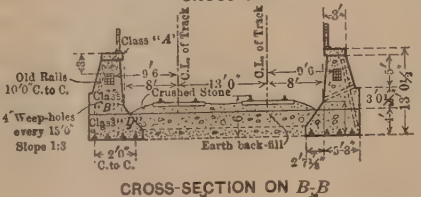


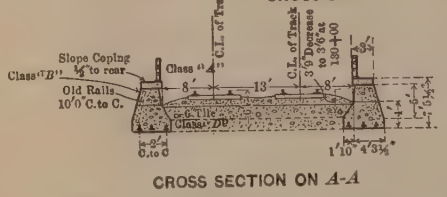
TABLE OF CONCRETE PROPORTIONS

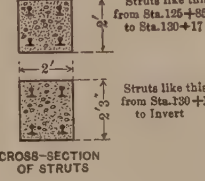
Class "A"	1	Cement,	2	Sand, 4 Gravel
" "B"	1	"	3	" 6 "
" "D"	1	"	4	" 7½ "

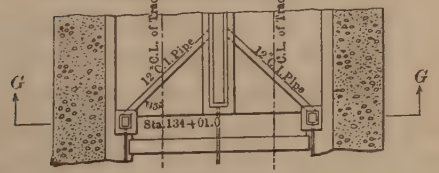


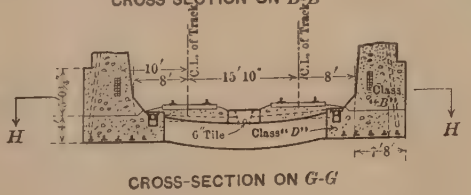












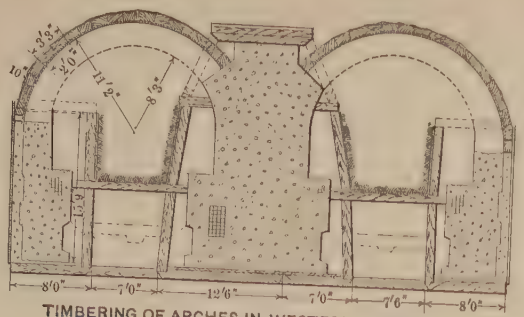
Notes:-
All Elevations refer to datum mean tide Sandy Hook, N.J.
All exposed corners and edges of concrete to be rounded to 1" radius unless otherwise specified.
Back of wall above weep-holes to be water-proofed with one coat of straight run coal-tar pitch ⅛" thick, Class "D"

DETROIT RIVER TUNNEL.
DETAILS OF WESTERLY OPEN CUT.

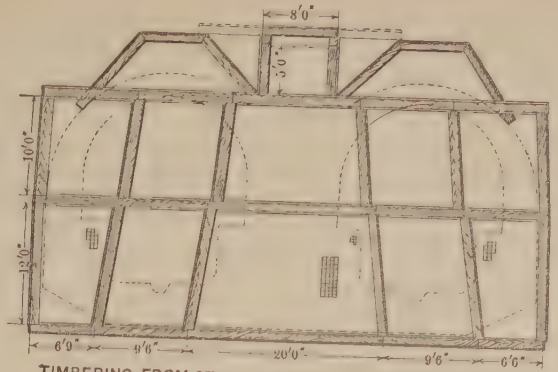
arches, and inverts, and provide working space, but would not include the full tunnel section. The modified elliptical sections were made necessary by the subsequent adoption of the shield as a means of making the excavation. The contractor planned originally to use the drift method for all approach tunnel work, but shortly after the work was started, it was found that, on account of the character of the clay, such a method was impracticable for building the full tunnel section, either in Windsor or in Detroit. The difficulties and problems as they presented themselves were somewhat different on each side of the river.

In Windsor the clay was so soft throughout that it flowed into the drifts almost before the timbering could be placed, and in some places it came in at the working face about as rapidly as it could be mined out. There was also considerable lateral movement of the drifts, due, possibly, to unequal loading of the ground surface, showing that the clay acted like a fluid and would have to be treated as such. At the time these conditions were encountered, one drift, on the Canadian side of the river, had been carried about 2 000 ft.—the bottom drift for the center wall—and work had been started on the top drift for this wall, but had to be abandoned. A cross-section of the center-wall drifts, as first planned, is shown on Fig. 3. It might have been possible to carry on the work in drifts under high air pressures, but the expense of such a method, if applied to the entire tunnel, was considered prohibitory. Later, 80 ft. of the Windsor Tunnel, near the river, was built entirely in drifts under compressed air. Conditions which will be described later rendered such action necessary and advisable. Except for this portion of the tunnel and a section near the east portal, where the work was done in an open trench, the character of the material was such that the contractor found it advisable to drive the excavation for the center wall first. The wall was then built, and served as a guide for the two side shields of full tunnel section, which followed closely thereafter.

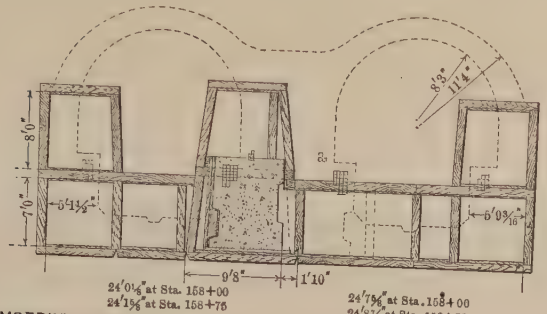
For the Windsor approach, a shield was used for the center wall, working above the bottom drift previously mentioned, and about 2 000 ft. of wall was built in this manner. This shield was of special design, was driven by hydraulic power, and excavation was made without the use of compressed air. The shield was abandoned at a point near where the center wall widens, approaching the subaqueous section, and the remainder of the wall was built in headings under compressed air.



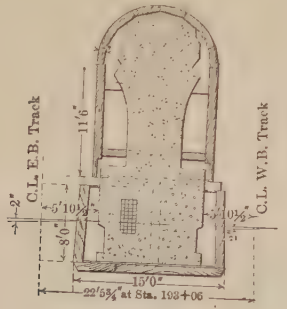
TIMBERING OF ARCHES IN WESTERLY APPROACH.



TIMBERING FROM STATION 185+43 TO STATION 186+21.



TIMBERING DETROIT APPROACH BETWEEN STA. 158+00 AND 158+75
SECTION AT STATION 158+75 LOOKING EAST.



TIMBERING OF CENTER
SHIELD DRIFT.

FIG. 3.

The movement of the center shield was very erratic at times, increasing with the depth of the overload, and had about reached its limit when abandoned.

The side shields were somewhat elliptical in shape, thus avoiding excessive increase in tunnel section. The cutting edge extended from the top of the center wall around to the bottom, including the arch, side-wall, and invert. The tunnel section was modified accordingly, as shown on Plates XXXVI and XXXVII, and the shield was built to conform thereto. The modified section is practically the same as the original, the effective thickness of ring having been maintained throughout. The side shields were used successfully, with and without compressed air, as conditions demanded.

The drift method was abandoned for the westerly approach tunnels, except for the center wall, but for reasons somewhat different from those demanding a change in design for the easterly approach. In Windsor the difficulties encountered affected only the work itself, with, perhaps, the single exception of a section where the tunnel passes under an overhead bridge, carrying Sandwich Street over the tracks of the Michigan Central Railroad. In Detroit a number of surface structures along the line of the tunnel had to be maintained, in addition to the main and yard tracks of the Michigan Central Railroad. While it cannot be said that a greater part of both tunnel arches could not have been built successfully under the drift method, yet there were sections where local conditions made such a method thoroughly impracticable. Experience with the first center-wall drifts showed that excavation in the drifts was invariably followed by considerable settlement of the ground surface above, due, undoubtedly, in a large measure, to the draining of the water from the clay. Experience proved that, if a number of drifts had been in operation at any one time, the settlement of the ground surface would have been a very serious matter, and the resultant property damages heavy.

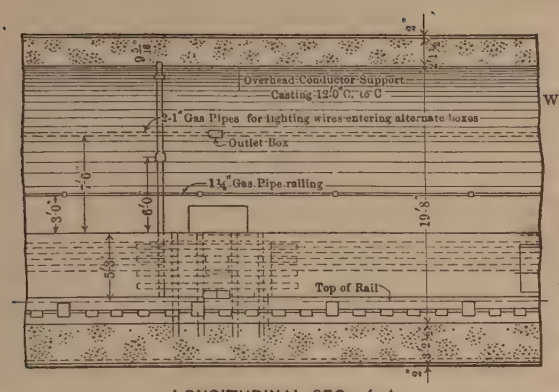
With the use of the shields, each working to the full tunnel section, the water-proofing was carried along as rapidly as the material was excavated, thus preventing seepage into the tunnel and consequent settlement of the ground surface. A portion of the westerly approach tunnel, near the portal, was built by the cut-and-cover method, which was also applied to a 400-ft. section in Windsor. A description of the approach tunnel work in detail follows.

Westerly Approach Tunnel.

The westerly approach tunnel extends from Station 137 + 40, the end of the open cut, to Station 158 + 75, about 60 ft. west of the dock line at the river, where connection is made with the subaqueous tunnel. For 1 375 ft. it follows, on a tangent, the northerly right-of-way line of the railroad to Station 150 + 15, from which point it curves to the right on a 2° curve to the river. This portion of the work involved 100 000 cu. yd. of excavation and 40 200 cu. yd. of concrete.

Excavation.—From Station 137 + 40, the station at the portal, to Station 139 + 60, the material was removed by steam shovel at the same time that excavation was in progress for the open cut. From Station 139 + 60 to Station 144 + 50, the center wall and east-bound tunnel section were built in open trenches, as was also the west-bound tunnel section from Station 139 + 60 to Station 141 + 80. The center-wall trench was first excavated and the wall built, then followed the side trenches, braced to the center wall. The west-bound trench was not carried beyond this point on account of the work involved in supporting a bridge abutment at Howard Street. The other trenches would probably not have been carried farther than the west-bound trench, had it not been for the previous construction in headings, between Stations 144 + 50 and 146 + 20, of a section of the east-bound tunnel, thus making it necessary to start the east-bound side shield as far east as Station 146 + 20. The west-bound shield was started at Station 141 + 80. Excavation for the center wall, east from Station 144 + 50 to the permanent shaft at Station 158 + 09, 66 ft. west of the connection with the subaqueous tunnel, was made in headings, part of which, east of Station 154 + 26, was done under compressed air. It will be noted from the profile that the top of the tunnel at the west portal is within about 2 ft. of the surface of the ground, and at Station 144 + 50, the east end of the open trench work, it is within about 8 ft., so that the deepest part of the trench was 30 ft.

Work in the headings was carried on at first from two shafts, one of which is the permanent shaft at Station 158 + 09, and the other a small temporary shaft at Station 145 + 60. Later, a small shaft was sunk at Station 154 + 26 for handling the work in compressed air, east from that station. The permanent shaft, known on the work as Shaft No. 1, was sunk by the Tunnel Company early in 1906, for the purpose of expediting the work, and before the contract was awarded. This

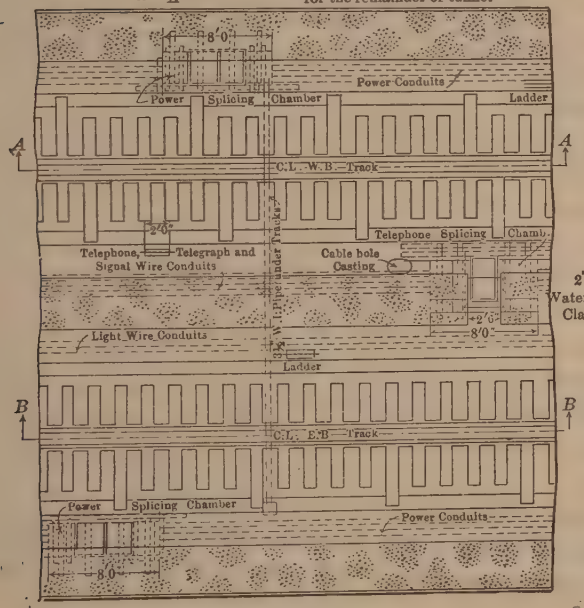

Overhead Conductor Support
Casting 12'0" C. to C.
2-1" Gas Pipes for lighting wires entering alternate boxes
Outlet Box
1¼" Gas Pipe railing
Top of Rail

LONGITUDINAL SEC. *A-A*

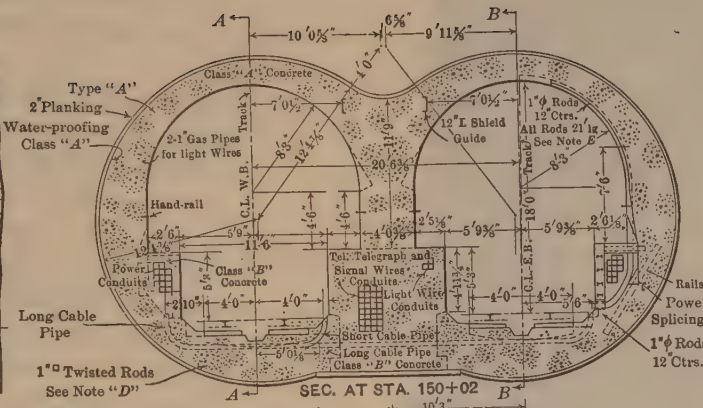
TABLE OF CONCRETE PROPORTIONS

Class "*A*"	1	Part Cement,	2	Parts Sand,	4	Parts Broken Stone or Gravel					
Class "*B*"	1	"	"	3	"	"	6	"	"	"	"	"
Class "*D*"	1	"	"	4	"	"	7½	"	"	"	"	"

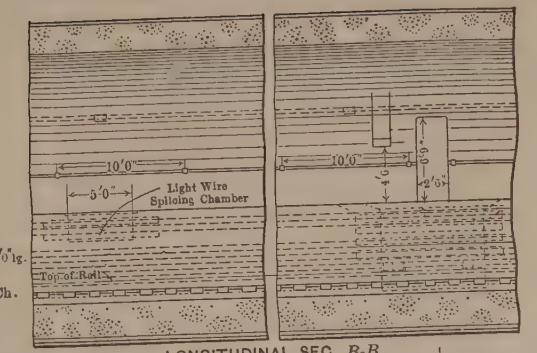
NOTE:
Class "*C*" Water-proofing from Subaqueous Tunnel to Shaft
" "*B*" " for 200 ft. West from Shaft
" "*A*" " for the remainder of tunnel



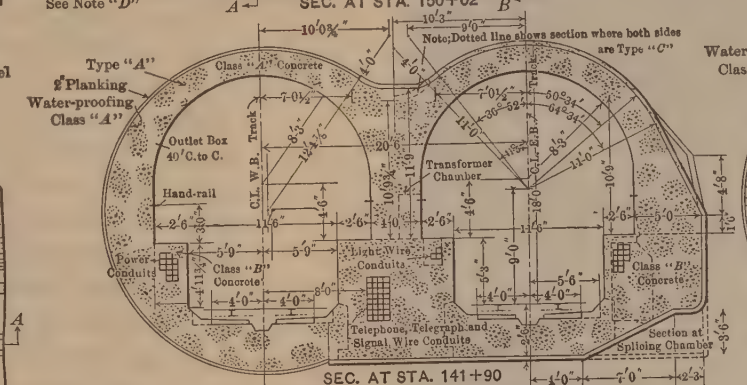
PLAN



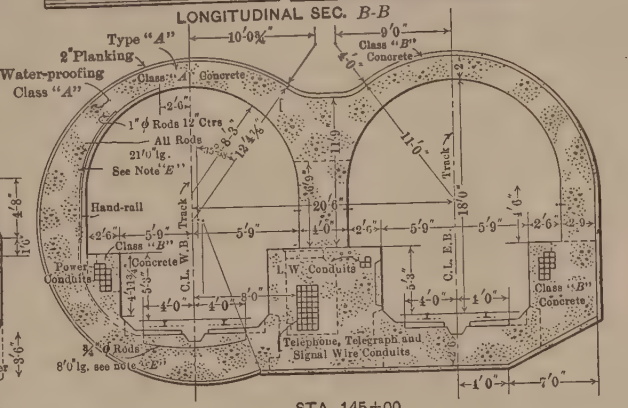
SEC. AT STA. 150+02



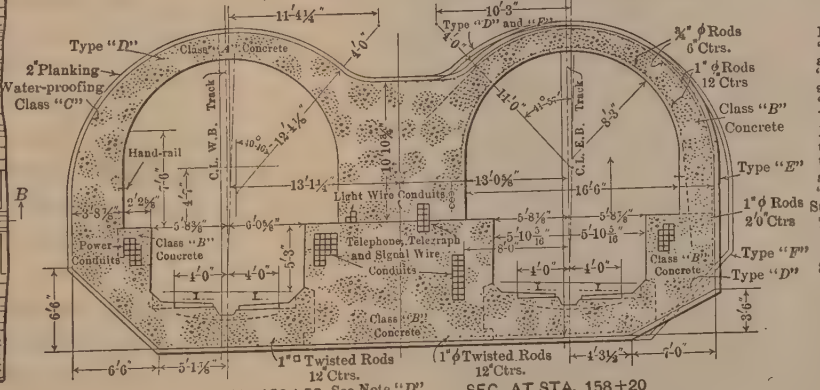
LONGITUDINAL SEC. *B-B*



SEC. AT STA. 141+90



STA. 145+00



SEC. AT STA. 158+53 See Note "D" SEC. AT STA. 158+20

Notes:
"*A*" Dotted lines at Sta. 141+90 show section when both W.B. and E.B. Tunnels are built according to Type "*C*"
"*B*" This section to be used for W.B. Tunnel only from Sta. 139+59 to Sta. 140+96 see section at Sta. 141+90 Type "*C*"
"*C*" Transverse reinforcement in invert rods ¾" ϕ rods placed under each tie in W.B. Tunnel Sta. 144+18 to 149+44 and in E.B. Tunnel Sta. 139+08 to 140+32 E.B. and Sta. 146+58 to Sta. 147+83 E.B.-Rods placed in center of space between ties, in W.B. Tunnel Sta. 137+40 to 141+81, Sta. 149+44 to 157+92 and Sta. 158+23 to 158+75 in E.B. Tunnel Sta. 137+48.5 to 139+08 E.B. Sta. 147+83 to 157+97 E.B. and Sta. 158+27 to 158+77.57 E.B.
"*D*" Longitudinal reinforcement in invert of 1" □ twisted rods, in W.B. Tunnel Sta. 137+40 to 141+81. Sta. 144+18 to 157+92 and Sta. 158+23 to 158+75, in E.B. Tunnel Sta. 136+48.5 to 140+32 E.B. Sta. 146+58 to 157+97 E.B. and Sta. 158+27 to 158+77.57 E.B.
"*E*" Reinforcement of North wall of 1" ϕ rods Spaced 6" C. to C. is put in from Sta. 142+66 to 148+50 W.B. and of South Wall from Sta. 149+93 to 151+40 E.B. on account of proximity of heavy retaining wall and bridge abutments.

DETROIT RIVER TUNNEL
DETAILS, WESTERLY APPROACH TUNNEL

shaft, as excavated, was 20 by 60 ft., covering both tunnels. When the contractor came on the work, in the fall of 1906, the shaft was turned over to him to be used as a base of operations and to be maintained and completed in accordance with the contract drawings. The first temporary shaft, known on the work as Shaft No. 2, was sunk by the contractors directly after they took over the work, and was just large enough to include the center wall.

The headings were started west from Shaft No. 1 and east from Shaft No. 2 at about the same time, early in the fall of 1906. Work from Shaft No. 1 was in two headings, one over the other, measuring, respectively, 9 by 9 ft. for the top and 11 ft. 6 in. by 10 ft. for the bottom. These headings were to provide for the building of the north half of that portion of the center wall below the top of the bench-wall. Fig. 3 shows a cross-section of these headings, the section of wall, and the method of timbering. Work in these drifts advanced to about Station 156, where it was discontinued, temporarily, owing to the settlement of the foundation piers of the Union Depot Company's viaduct, which carries three tracks, crossing the line of the tunnel just west of Station 156. These drifts were carried east from Shaft No. 2, the top, middle, and bottom center drifts to provide for the building of the full section of the center wall. Fig. 3 also shows a general cross-section of these drifts, the method of timbering, and a cross-section of the wall. The timber bents were 2 ft. from center to center, and the clay at the sides was retained by 2-in. wooden sheeting placed horizontally outside of the bents. As stated, excavation for the entire center wall from Station 144 + 50 to Shaft No. 1 was made in this manner, and the portion east of Station 154 + 26, except the first drifts west from Shaft No. 1, was done under compressed air. From Station 154 + 26 to Station 155 + 16, where the ground was especially soft, owing to the proximity of one of the test-pits which had stood full of water for some time, a pressure of 17 lb. was required. East of this, to Shaft No. 1, the work was under a pressure of about 7 lb. Before any work was done under the Union Depot Company's viaduct, that structure was supported temporarily on concrete piers sunk below the bottom of the tunnel. This work was done by the contractor, and will be described later.

Previous to the installation of the side shields, the section of the east-bound tunnel between Stations 144 + 50 and 146 + 20 had been built in drifts, the general scheme being as outlined on Fig. 3.

The east-bound shield was installed at Station 146 + 20 and the west-bound shield at Station 141 + 80, as before noted, and the excavation for the full tunnel section on either side of the center wall was made with the shields from these points down to Shaft No. 1.

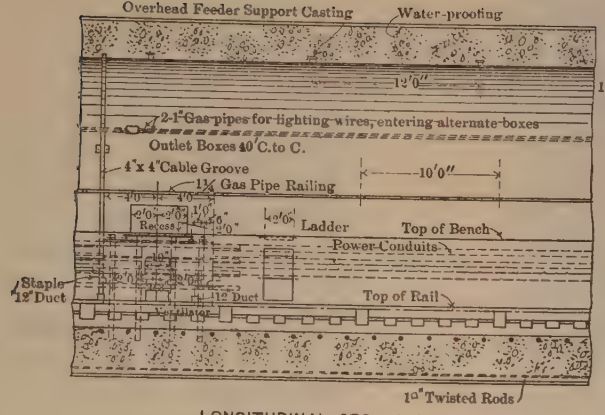
At about the time that work was started west from Shaft No. 1, headings were also started east from the shaft toward the river. The full section of both tunnels, 2 ft. east from and adjoining the shaft, the north half of the center wall, below the top of the bench-wall, and a short piece of both east- and west-bound side-walls, up to the level of the bottom of the ducts, had been built when the work was temporarily abandoned on account of a change in the general method of handling it. It was originally intended that the westerly approach tunnel should be completed in drifts to the point of connection with the subaqueous tunnel. This, however, was found impracticable without the aid of compressed air, on account of the marked tendency of the clay to flow. At that particular time the contractor did not contemplate using compressed air on any part of the work in Detroit, but had decided to complete the section of tunnel from Shaft No. 1 to the connection with the subaqueous tunnel in the open by a coffer-dam.

The clay in the drifts and back of the shields was taken out by hand, with the aid of a semicircular knife, loaded into narrow-gauge cars, and cable-hauled to the shaft, where it was elevated to the surface and dumped on standard-gauge flat-cars. The semicircular knife was also used in the open-trench excavation. For such work it was drawn by a cable from a hoisting engine, and was easily guided by one man. The material from the trenches was loaded in skips and hoisted by derricks to flat-cars.

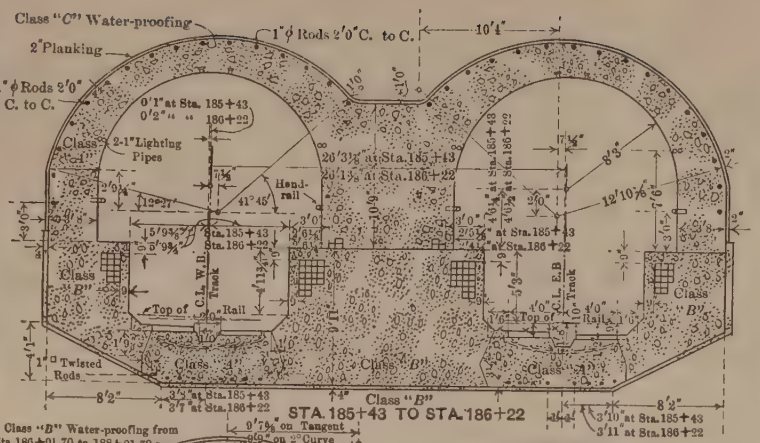
In the drifts the usual rate of progress of excavation for 24 hours, under normal conditions, was about 12 ft. The average rate of advancement of the side shields for working days was about 10 ft. per day of 24 hours.

The average cost of excavation in the drifts, where the clay was fairly stiff, was $2.37 per cu. yd. Under compressed air in the drifts, the average cost was $3.50 per cu. yd.; with the shields, it was about $4.90 per cu. yd. These figures include the cost of timbering, depreciation of plant, and distributive share of general expense.

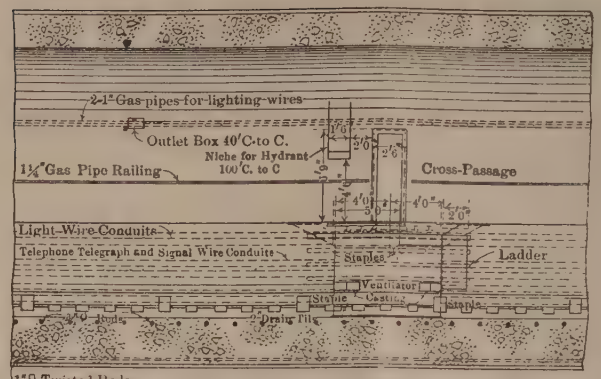
Shields.—A description of the side shields, the method of timbering back of the shields, and the water-proofing is given here, as the general methods were the same on each side of the river. The side



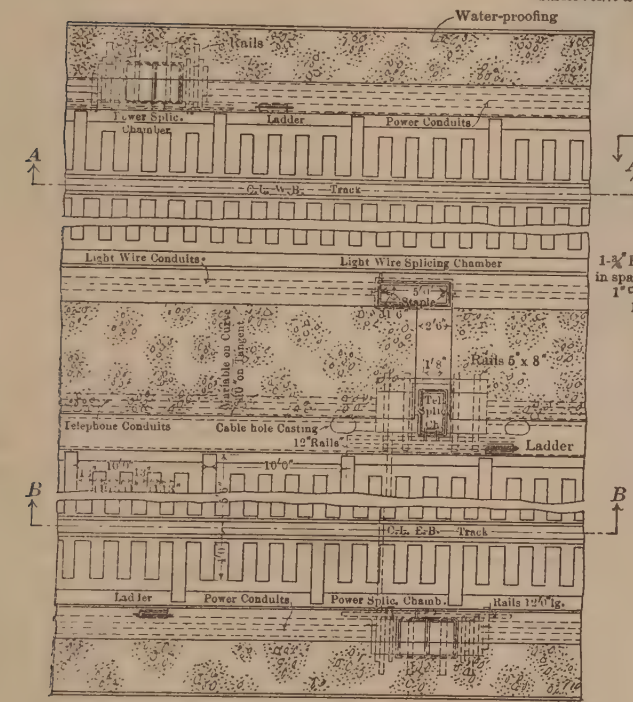
LONGITUDINAL SEC. *A-A*



STA. 185+43 TO STA. 186+22



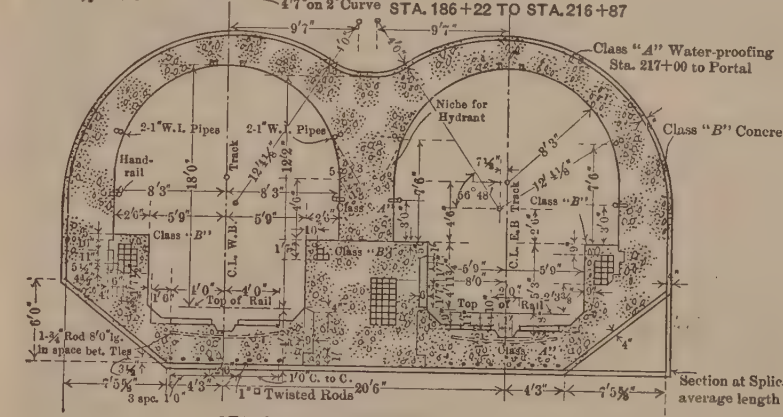
LONGITUDINAL SEC. *B-B*



PLAN

Class "B" Water-proofing from Sta. 186+01.70 to 188+01.70
Class "A" Water-proofing from Sta. 188+01.70 to 217+00
2" Planking
Class "A"
Class "B"
Class "B"
Class "A"
Old Rails 6'0" lg. 1'6" at 12'0" Ctrs.
12" E Shield guide
1-¾" Rod 8'lg. in space bet. ties
1" □ Twisted Rods 12" C. to C.
3'3¼" on Tangent
2'11¼" on 2° Curve
4'9⅛" on Tangent
4'7" on 2° Curve

STA. 186+22 TO STA. 216+87



STA. 216+87 TO WINDSOR PORTAL

General notes:

All exposed corners and edges of concrete to be rounded to 1" radius unless otherwise specified.

Outlet boxes for lighting to be 40' apart on each side of each tunnel, staggered.

Ladders to be 50' apart on each side of each tunnel staggered.

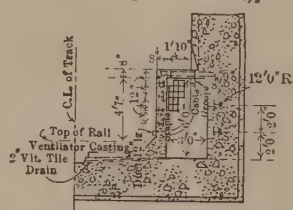
Splicing chambers to be 400' apart and cross-passages 200' apart.

Dimensions marked (x) vary for high tie and are *D* x 1" less than same dimensions for low tie when *D* = degree of curvature of track.

There are 40 telephone ducts from telephone splicing chamber at Sta. 186+15 to Windsor Shaft and 32 from Windsor Shaft to Windsor Portal.

TABLE OF CONCRETE PROPORTIONS

Class "*A*"	1	cement,	2	sand,	4	broken stone or gravel.			
" "*B*"	1	"	3	"	6	"	"	"	"
" "*D*"	1	"	4	"	7½	"	"	"	"



Section through Power Splicing Chambers Sta. 217+91 W.B and Sta. 217+95 E.B.

DETROIT RIVER TUNNEL
DETAILS OF EASTERLY APPROACH TUNNEL

shields for Detroit and Windsor were of the same design, but of slightly different dimensions, to conform to the design of the tunnel sections, as shown by Figs. 4 and 5. The shields were large enough to include a ring of timbering 10 in. thick outside of the concrete section. Fig. 4 shows the plan of the Windsor shield, and Fig. 5 is a sketch in perspective of the shield and center wall, showing the timbering for the center wall excavation and that back of the shield. The shields were 13 ft. over all. The outside shell was of $\frac{3}{4}$-in. steel plate, strongly reinforced with angle-irons. The horizontal and vertical reinforcement was arranged so as to leave six pockets from which to remove the clay. The main body of the shield was 7 ft. 6 in. long, with a heavily reinforced cutting edge, to which was attached a tail-plate of $\frac{1}{2}$-in. steel plate, 4 ft. 2 in. long. The tail-plate was not reinforced, its only duty being to hold the clay until the timbering could be placed. Each shield was supplied with twenty-one 5-in. hydraulic jacks, capable of expanding 20 in. The jacks were designed to be used at a pressure of 5 000 lb. per sq. in. and, in actual practice, were used up to 8 000 lb. Later, the two bottom jacks were replaced by two 8-in. jacks, and an extra 5-in. jack was added next to the center wall at the top. The shield traveled along the center wall, bearing against the channel irons, as shown in Fig. 5.

A temporary floor or working platform was built behind the shield, at the level of the top of the bench-wall, and carried a track for handling the material, the excavated material being loaded on cars by a mechanical conveyor operated by compressed air.

The clay was cut, as before noted, with semicircular knives pulled by two men and guided by a third. The excavation was carried about 2 ft. ahead of the shield over practically the full tunnel section, and the shield was shoved forward the 2 ft. by the hydraulic jacks, bearing against the timbers already placed. The timbering consisted of pieces 4 ft. long, 10 in. deep, and of variable widths, cut to segments of a circle, and placed as shown in Fig. 5. When the shield was shoved ahead, the timbers (or cants, as they were called) were placed inside the tail-piece, as many rings being placed at each move as were required to fill in the distance covered by the advance of the shield. The cants in each ring were placed so as to break joints with those in the preceding ring. Each ring was expanded to the full radius against the tail-piece by wedges forced between several of the butting cants by the jacks.

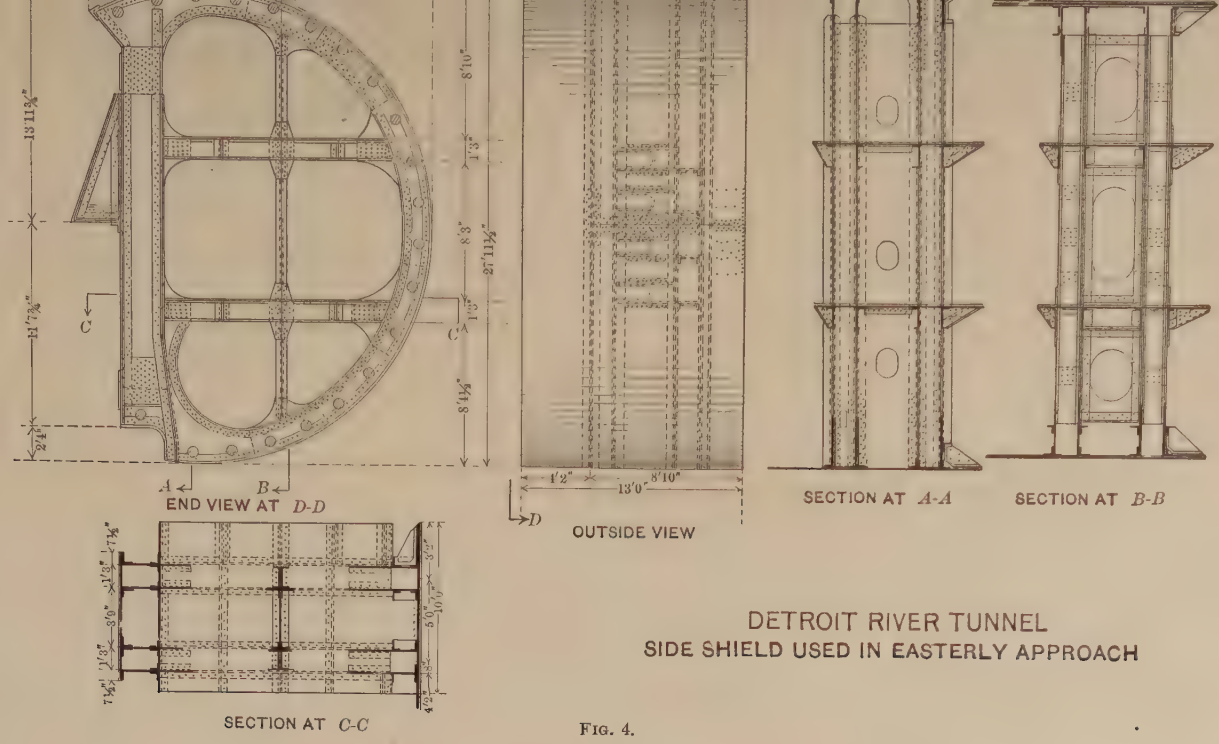

END VIEW AT D-D
OUTSIDE VIEW
SECTION AT A-A
SECTION AT B-B
SECTION AT C-C
DETROIT RIVER TUNNEL
SIDE SHIELD USED IN EASTERLY APPROACH

FIG. 4.

FIG. 1.—INTERIOR OF APPROACH TUNNEL. ARCH FORM IN POSITION.

FIG. 2.—SIDE SHIELD IN OPERATION.

There was very little trouble in keeping the shields on line, but they were off grade frequently. The variation from true grade amounted at times to 6 in., and seemed to occur at intervals, regardless of the care exercised in operating the shields. The irregularity in the grade line followed was due, in no case, to a variation in the character of the material encountered. The shields were never quickly brought back to true grade line when once started on a downward or upward path. This tendency is natural enough with any shield, but it was probably aggravated on this work owing to the design of the shield, which obviously was not as rigid as one with a cutting edge describing a full circle or ellipse, with jacks on all sides.

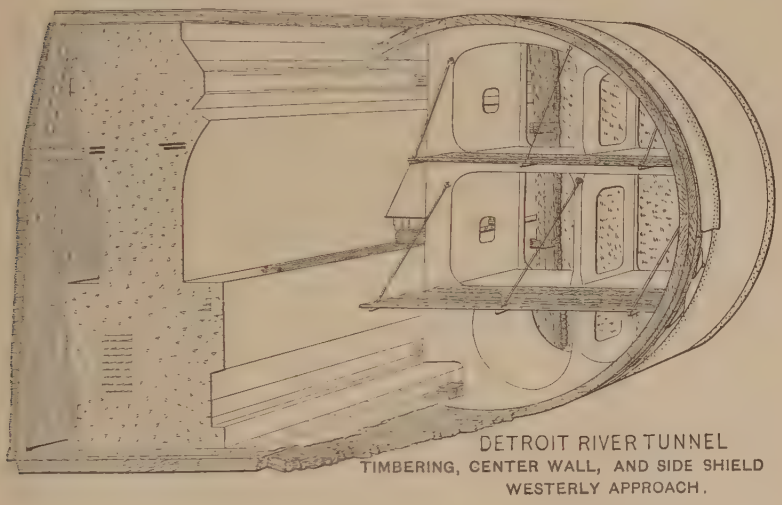


FIG. 5.

Views of work with the shield in the approach tunnel are shown on Plate XXXVIII.

Water-proofing.—The water-proofing in the approach tunnels consists of alternate layers of felt and coal-tar pitch. There were three classes of water-proofing, made up as follows: (A) three layers of felt and four of pitch; (B) five layers of felt and six of pitch; and (C) ten layers of felt and eleven of pitch.

Two general methods were used in making the laps between the successive layers of felt and the successive sections of the water-proof sheet, as follows: (1) Where it was possible, the successive layers of felt were laid so that the vertical or transverse edge, as the case might

be, of each strip of felt would step forward one-third the width of the preceding strip, or one-fifth, or one-tenth, according to the number of layers. (2) The water-proofing was applied in three, five, or ten successive layers, with a 3-in. lap between the strips of felt, breaking joints between layers. It was necessary to use this method with the ten-ply water-proofing, when the working space was limited, and there was not room to continue the first method of lapping for any considerable distance, where, for instance, the concrete followed the water-proofing up to the limits of the space excavated.

One method would probably have been as effective as the other under favorable conditions, but it was found that back of the shields, where high temperatures obtained, due to the setting of the concrete, and especially under compressed air, it was difficult to hold the water-proofing in place on the sides and roof of the tunnel, and prevent the felt from sagging, without nailing at least three layers to the timbers. With the first method, three layers, or as many as desired, could be nailed soon after the first felt had been applied. With the five-ply, three layers were nailed to the timbers. The objection to the second method was, that, under high temperatures, the first layers placed sagged considerably before the nails could be driven. Tests were made in an effort to hold the water-proofing by nailing only the first layer, so that the nail-heads would be covered by at least two layers of felt. Where there was not much heat, this method worked very well, particularly where the first layer was of burlap. Under high temperatures, however, it was necessary, as stated, to drive nails through the three layers and rely on a coating of pitch over the nail-heads. If there were any weak places, they did not appear during the progress of the work, and were not expected to be noticeable where the work was under compressed air. As a reinforcement to the nailing process, a strip of tin was used, which followed the edge of each strip of the outer layer of felt, thus giving continuous support to the felt and acting as a washer for the nails.

From the invert upward and over the arch, the water-proofing was carried in three lifts, corresponding to the lifts of concrete. Where there were only three layers of paper, the horizontal joint between two lifts was made by a 12-in. lap of the sheet of three layers of the upper lift over the outside of those of the lower; where there were five thicknesses, three of the upper lifts lapped 12 in. outside those of

the lower and two inside. With ten thicknesses, the bottom lift was separated sufficiently for a 12-in. lap with two sheets of five layers each; four layers of the upper lift lapped 12 in. over the outside of the lower, the next three layers being inserted between the two sheets of the lower, and the inside three lapped inside. This latter arrangement is assuming the water-proofing to have been placed on the forms in advance of the concrete. If laid on the concrete, as in the open trenches, the words "inside" and "outside" should be interchanged. The vertical, or transverse, joints between successive sections of the water-proof sheet along the tunnel were made as by the first method, or as described for horizontal joints, depending on the conditions.

There was some difficulty in making the lap between old and new water-proofing where considerable time had elapsed between the placing of the first and last, owing to the felt having become water-soaked and more or less rotten. This trouble was overcome to a great extent by using burlap for the lap in place of the felt in the first water-proof sheet and soaking the burlap in creosote. There was also some trouble at first from the water-proofing on the center wall, placed in the drifts, being torn by the edge of the shield as it passed. On the portion of the wall built after the shields were started this trouble was eliminated by rolling up the loose edge of the water-proofing on the center wall and protecting it by a covering of boards.

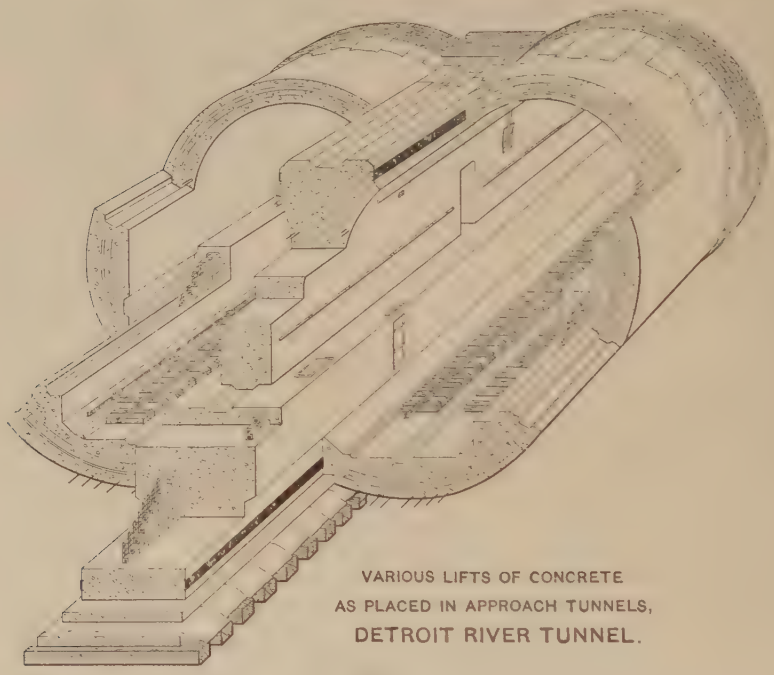
The coal-tar pitch was used at a temperature of not less than 250° Fahr. If the surface on which the pitch was to be applied was damp, and it was not practicable to dry it, an extra layer of felt was used.

Concrete.—In the approach tunnels the concrete was placed in lifts, as shown by Fig. 6, which represents the shield section, but shows in general how the concrete was carried up throughout the approach tunnels. The standard length of run was 12 ft., and the concrete in each run was allowed to set sufficiently for the form to be moved before the succeeding run was made.

In the westerly approach, the concrete for the center wall, up to the joint just below the top water-proofing, was deposited from dump-cars run along the top drift. The concrete above this, except on the work under compressed air, was deposited from the ground surface above through chute holes, 12 in. in diameter, at intervals of about

12 ft. The top drift, for the work under compressed air, was made higher than for the work in normal atmosphere, so that the concrete for the entire wall could be delivered in cars. Chute holes, 12 in. in diameter, were also provided at intervals of 12 ft. over both the east- and west-bound tunnels in the westerly approach, for depositing concrete for the invert, side-wall, and arch, back of the shield.

In the approach tunnels, two classes of concrete were used: 1:2:4 and 1:3:6. The first constituted the main body of the tun-



VARIOUS LIFTS OF CONCRETE
AS PLACED IN APPROACH TUNNELS,
DETROIT RIVER TUNNEL.

FIG. 6.

nel, the latter being used only over the ducts in the bench-walls and outside of the water-proofing, as a protection course. It was omitted back of the shields, where the water-proofing was applied on the timbering. Still another grade, not indicated on the plans, was used at the connection of the westerly approach with the subaqueous tunnel, outside of the water-proofing, to insure a water-tight connection. This consisted of one part of cement, one of sand, and two of gravel. The gravel for all concrete used in the tunnel was taken from the Black

River, at Port Huron, Mich., by "sand suckers," these boats making direct delivery for storage or immediate use. The gravel was screened on the work, thus giving the proper proportions of clean sand and gravel.

The forms for the concrete on the approaches consisted of steel or wooden ribs, with 2-in. wooden lagging covered by heavy sheet iron to give a uniformly smooth surface.

Maintenance of Structures.—As has been stated, the tunnel lies wholly within the Railroad Company's right of way, and, in Detroit, the open cut and 1 258 ft. of the tunnel are parallel with, and immediately adjoining, the northerly right-of-way line. On account of the limited width of the right of way along this part of the work, and the necessity for retaining as many tracks in service as possible, the first track south of the open cut—the west-bound passenger track—was supported on piles. The pile support was continued under this track to Station 145 + 00, along the open-trench work.

Along the northerly right-of-way line are the abutments of three bridges, carrying Howard Street, Lafayette Avenue, and Twelfth Street, over the Michigan Central Company's tracks. These abutments settled from 8 to 12 in., but no serious damage resulted. There is also a large five-story brick building on the northerly right-of-way line, east of Layafette Avenue, along which steel sheet-piling was driven to the level of the bottom of the tunnel, hoping to guard against settlement. The building settled, however, about 6 in. on the side next to the tunnel and ½ in. on the opposite side, 100 ft. back of the right of way. On the opposite, or south, side of the right of way, 40 ft. south of the tunnel, a large eight-story building showed no settlement. This building was constructed while the tunnel work was in progress, before excavation with the side shields had been made, and the foundation was carried down to within 7 ft., approximately, of the grade of the bottom of the tunnel. The south abutment of the Fort Street Bridge, which is 25 ft. south of the tunnel, settled 12 in., but caused no serious damage.

The viaduct of the Fort Street Union Depot Association, previously referred to, which crosses the line of the tunnel near Station 156, was carried over the tunnel, temporarily, by six concrete piers, three under each side of the viaduct, which the contractor sank to a depth below the bottom of the tunnel, as shown by Fig. 3, Plate

XXXIX. These piers settled a maximum of 0.3 ft. The concrete wall of the tunnel, which encloses the lower part of two of the piers, settled 0.1 ft. with these piers. The columns of the viaduct, which rested on 24-in. I-beams carried by the piers, were jacked up from time to time, thus maintaining the grade of the viaduct.

A 42-in. water main in River Street, on which street the above-mentioned viaduct is located, was temporarily supported by steel hangers swung from the viaduct.

A double 5-ft. sewer, crossing the line of the tunnel at River Street at a very flat angle, was temporarily replaced by a steel tube and afterward rebuilt of reinforced concrete.

At Twelfth Street a 5-ft. single brick sewer, was temporarily replaced by a steel pipe. Part of the tunnel at this point was built in open trenches or cut and cover, the pipe being carried by trusses over the trenches.

The settlement of the tracks over the tunnel was not very great; it was gradual, and was taken care of from time to time by re-ballasting.

Easterly Approach Tunnel.

The easterly approach tunnel extends from Station 220 + 58 to Station 185 + 43. It was originally intended that the limits should be between Stations 217 + 06 and 185 + 00, but, after the work in the initial drifts had been in progress for some time, the character of the clay, as disclosed near the upper end, was such that it was decided to extend the tunnel section farther east than first proposed. Experience in the headings near the river showed that it would be better, on account of the softness of the clay, not to carry the headings farther west than Station 185 + 43, and to make the connection with the subaqueous tunnel at that point.

Work on the easterly approach tunnel was conducted at first from two shafts, one, the proposed permanent shaft, 20 by 60 ft., including both tunnels; the other, a temporary working shaft, just large enough to enclose the center-wall headings. Subsequently, two other temporary shafts were sunk, similar to the first. The proposed permanent shaft, known as No. 1, was located at Station 185 + 93, and the temporary shafts, known as Nos. 2, 3, and 4, at Stations 192 + 97, 203 + 50, and 216 + 80, respectively. The construction of the easterly approach tunnel involved 196 700 cu. yd. of excavation and 65 200 cu. yd. of concrete.

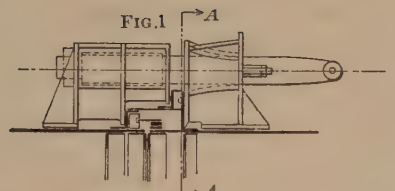


FIG.1

PIN CONNECTION AT TUBE SPLICE.

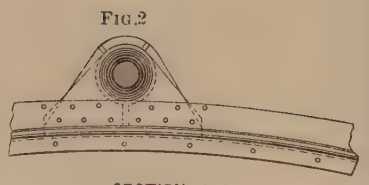
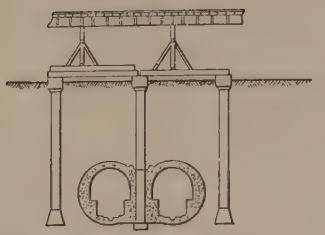


FIG.2

SECTION *A-A*



SUPPORTS FOR RIVER ST. VIADUCT.

FIG.3

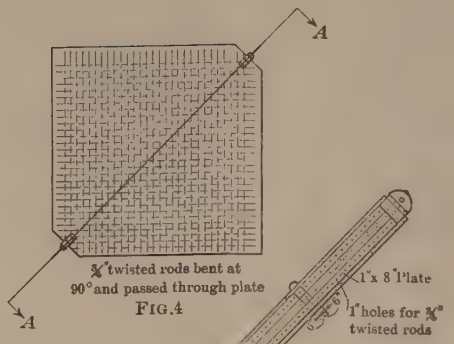
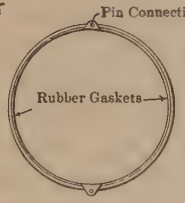
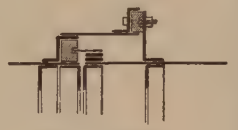


FIG.4

CONCRETE ANCHOR

RUBBER GASKETS





JOINT AS MADE BY DIVER

FIG.5

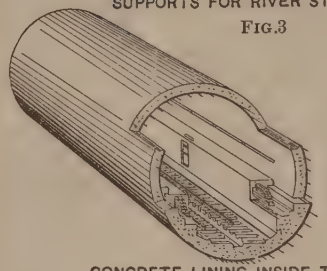
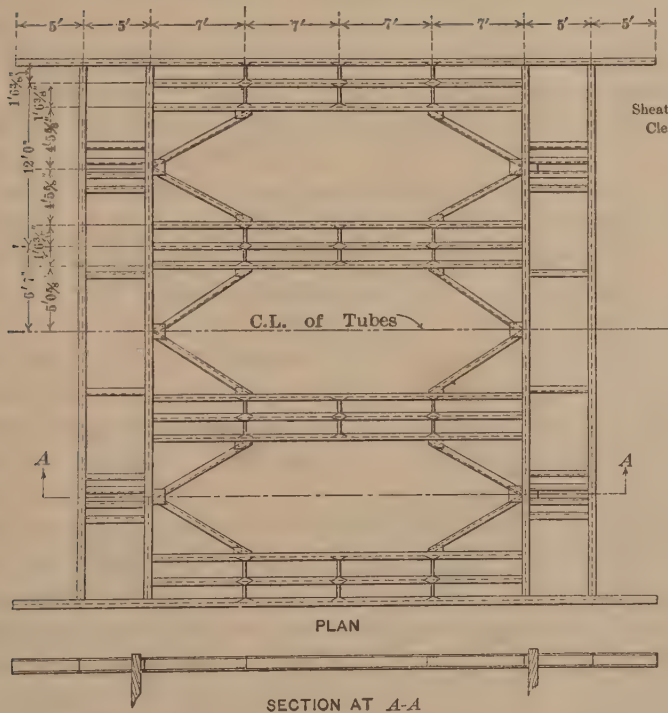


FIG.6

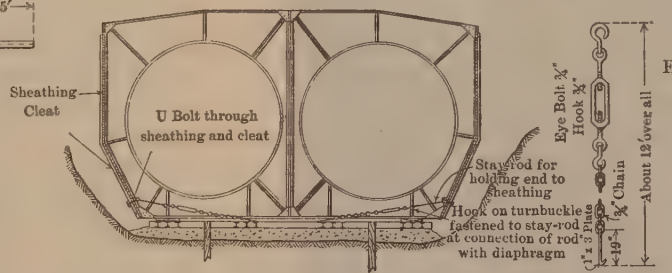
CONCRETE LINING INSIDE TUBES



PLAN

SECTION AT *A-A*

FIG.7


Sheathing
Cleat
U Bolt through sheathing and cleat
Stay-rod for holding end to sheathing
Hook on turnbuckle fastened to stay-rod at connection of rod with diaphragm

METHOD OF ANCHORING END OF TUBE SECTION TO GRILLAGE. NATURE OF DIAPHRAGM.

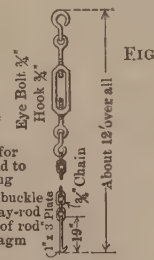
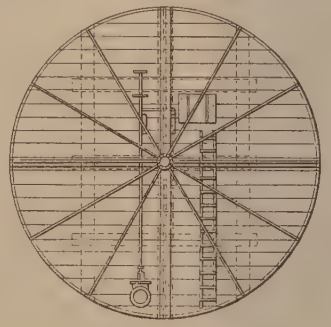


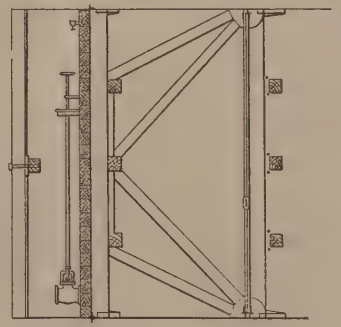
FIG.8

SKETCH OF TURNBUCKLE ARRANGEMENT

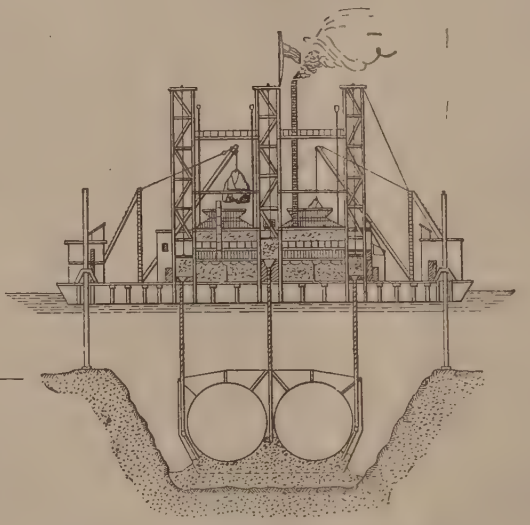


END VIEW OF BULKHEAD

FIG.9



LONGITUDINAL SECTION ON C.L.



METHOD OF PLACING CONCRETE BY TREMIE.

FIG.10

DETROIT RIVER TUNNEL.
DETAILS OF CONSTRUCTION.

Excavation.—The first excavation for the easterly approach, started in the fall of 1906, was in a single heading east from the proposed permanent shaft and in the corresponding heading west from Shaft No. 2 to meet that from Shaft No. 1. This opening was just large enough to include the north half of that portion of the center wall below the top of the bench. To provide for the handling of concrete for this part of the wall, another small drift was put through between the two shafts over the first drift and the completed section of wall. This portion of the wall was built before excavating was begun for its south half, because of the great pressure from the clay and the danger of collapse of the drifts, in case too large openings were made simultaneously. Later, the bottom drift for the south half of the center wall, called the south bottom center drift, was put through and the concrete was placed up to about the level of the top of rail. Great difficulty was experienced in this drift on account of the side pressure and the swelling of the clay from the face and roof. It seemed at times as though the drift would have to be abandoned. There was also a slight movement of the north half of the wall already built, as this drift was opened up, and a large settlement crack, parallel to the drift, appeared in the ground surface, attended by a settlement on top of from 1 to 2 ft.

At about the same time that work was being carried west from Shaft No. 2, the bottom center drift was also being carried east from this shaft. This was a double drift for the full width of the center wall. (It will be noted that the center wall becomes narrower toward the east, owing to the divergence of the center lines of the tunnels, from 26 ft. 4 in. at the river to 20 ft. 4 in. at Station 197 + 37, the point of tangent.) Excavation for the bottom center drift was also pushed from Shafts Nos. 3 and 4 as soon as they were built, which was early in 1907. Excavation was started for the middle center drift east from Shaft No. 2, but had to be abandoned, as the clay came in faster than the timbers could be placed. The middle center drift, west from Shaft No. 2, was also started—over the south bottom center drift—but likewise had to be abandoned. Cross-sections of these drifts were practically the same as those shown by Fig. 3 for the westerly approach tunnel. The timber bents in these drifts, however, were about 12 in. from center to center.

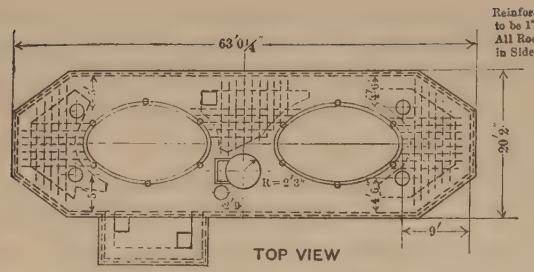
The center shield, previously mentioned, was installed at Shaft No. 4 in June, 1907. It was thought that the excavation for the full

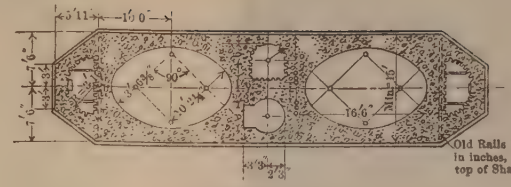
section of center wall above the bottom drift already excavated could be completed with this shield, without the use of compressed air. The shield advanced to Station 196 + 60, where it was taken out, as it was found impossible to keep it in position. The remainder of the center-wall excavation, from the point of removal of the center shield to the work at Shaft No. 1, was made in drifts under compressed air. A cross-section of the shield drift is shown on Fig. 3; the center drifts under compressed air were practically as shown there, but the top drift was made higher than is shown by Fig. 3, as that represents the section built without the use of compressed air. As noted, the higher drift was to provide for the delivery of concrete in cars for the entire wall. It will be remembered that, where compressed air was not used, concrete was deposited for the upper part of the wall through chute holes put down from the surface.

As previously stated, the building of the south bottom center drift, between Shafts Nos. 1 and 2, was attended by considerable disturbance of the ground surface and clay overlying the tunnel. As a result of this disturbance the timbering in Shaft No. 1 shifted position in such a manner that it was necessary to widen the shaft in order to provide clearance for the completion of the permanent shaft. The work of widening and re-timbering had reached a point about half way to the bottom of the shaft, when, owing to an unfortunate combination of working conditions, together with a very heavy rainfall, the surrounding clay was found to be moving, bringing such pressures on the timbers as to cause a total collapse of the shaft. This was in the fall of 1907. The shaft was abandoned permanently, and work on this part of the tunnel was discontinued until early in 1908.

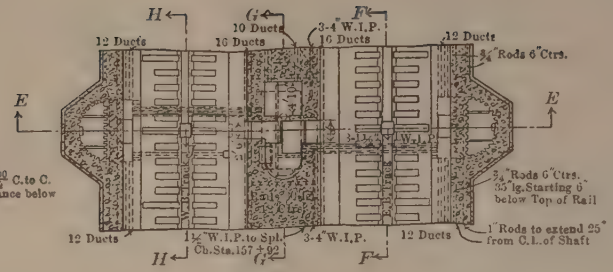
Prior to the failure of the shaft, two bottom headings and one middle heading had been carried through to Shaft No. 2, with an extension of five additional bottom headings 12 ft. east, and six a like distance west from the shaft. When work at this point was resumed, it was under compressed air, as all previous experience with the clay at Windsor had shown the impracticability of carrying on the work in drifts without it. The full section of tunnel from Station 186 + 23 to Station 185 + 43 was built in drifts, a cross-section of which is shown by Fig. 3. Provision was also made at this time for completing the center-wall drifts between this work and Shaft No. 2 under compressed air. For the work near Shaft No. 1, a

DETROIT RIVER TUNNEL.
DETAILS OF DETROIT SHAFT

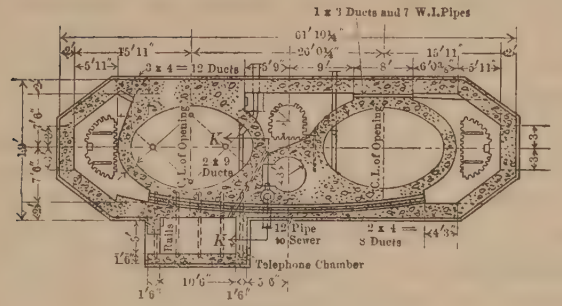

Reinforcement in Concrete Covers to be 1" dia. Rods, Spaced 12" apart All Rods are to be embedded 1 foot in Side-walls.
63'0¼"
20'2"
R=2'3"
9'
TOP VIEW



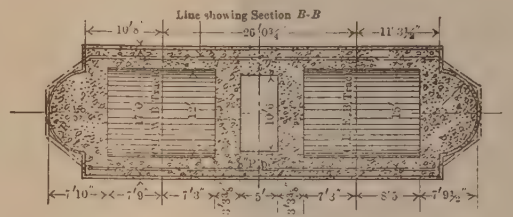
SECTION *B-B*



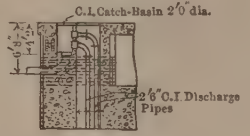
SECTION *C-C*



SECTION *A-A*



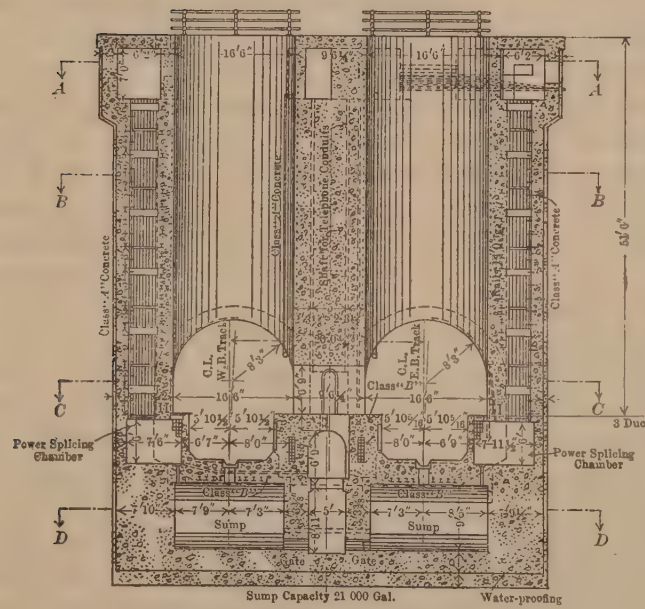
SECTION *D D*



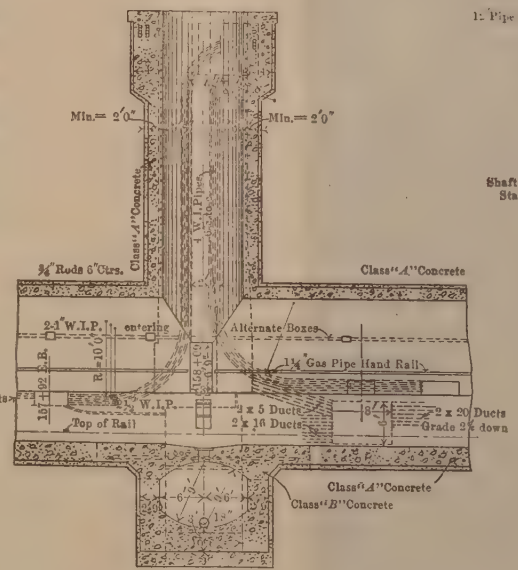
SECTION *K-K*

TABLE OF CONCRETE PROPORTIONS
Class "*A*" 1 Cement, 2 Sand, 4 Broken Stone or Gravel
Class "*B*" 1 " 3 " 6 " " " "

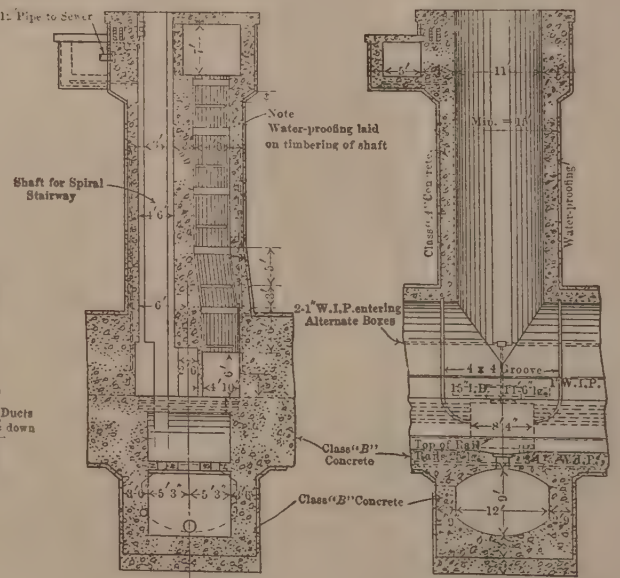
General Notes
All exposed corners and edges of concrete to be rounded to 1" radius unless otherwise specified.
Spiral Stairway furnished and placed under another contract.
Use Class "*A*" water-proofing above water level.
Class "*B*" between water level and track level Class "*C*" below track level.
All stationing W.B. except where noted.



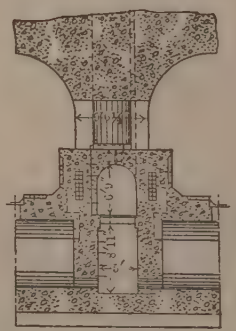
SECTION AT C.L. OF SHAFT



SECTION *F-F*



SECTION *G-G* SECTION *H-H*


SECTION *J-J*

material shaft, 5 ft. in diameter, with a material lock at the top, 13 ft. in diameter, was built just west of the old shaft. The new shaft was sunk on the center line of the center wall. For the work west of Shaft No. 2, a material and man lock was placed in the drifts just west of the shaft. An engineers' shaft, 3 ft. in diameter, with a lock at the top, was sunk about 12 ft. east of the old shaft, over the east-bound tunnel. It was several months after the work between Shafts Nos. 1 and 2 was placed under compressed air that the center shield was abandoned, and the center drifts east of Shaft No. 2 were similarly treated.

The excavation for the full tunnel section on either side of the completed center wall, from Shaft No. 4 down to Station 186 + 23, 30 ft. east of old Shaft No. 1, was made with the side shields. An attempt was made to operate these shields without compressed air, but was found to be impracticable. The clay flowed in at the face of the shields, just as it had in the drifts, and air-locks were placed about 200 ft. west of Shaft No. 4. All work thereafter was done under compressed air, the pressure required ranging from 10 to 20 lb., the higher pressures being near the river.

The tunnel section east of Shaft No. 4, known as the approach tunnel extension, was built by cut-and-cover methods, except a portion of the center wall below the top of the bench-wall, which was first built in drifts to provide a means of bracing the trenches.

A new permanent shaft was located and built at Station 187 + 68, which provides an opening over each tunnel. This shaft is substantially of the same design as the Detroit shaft, details of which are shown on Plate XL.

The rate of progress of excavation at Windsor, with the shields and in the drifts, under favorable conditions, was about the same as at Detroit, being approximately 10 ft. per day of 24 hours with the shields and 12 ft. per day in the drifts. The maximum distance covered by one of the shields in a single day was 19 ft. 10 in.

The cost of excavating in drifts without compressed air averaged about $2.40 per cu. yd. under favorable conditions; with compressed air, the cost was about $3.50 per cu. yd., except for the work near old Shaft No. 1, where the cost ran as high as $16 per cu. yd. With the shields the fair average cost was $4.90 per cu. yd. These costs include timbering, compressed air, and incidentals. The cost of the

compressor plant for supplying air to two side-shield headings and two center-wall headings was a trifle more than $80 per day.

Concrete.—In the easterly approach tunnel the concrete was placed in the same manner as for the westerly approach, except that all concrete was delivered in dump-cars in the tunnel. Arches were filled by shoveling the concrete into the form from the end, as the form for a 12-ft. section was set up complete before concreting was started. Concrete for the center wall, back of the center shield, was delivered in cars up to a level just below the water-proofing course, and above that through chute holes put down from the surface at intervals of about 30 ft. All concrete for the portion of the center wall built under compressed air was delivered in the drifts in cars, as previously stated. The average cost of the 1:2:4 concrete was about $8.25 per cu. yd. in the side-shield headings, and a little more than $6 per cu. yd. in the center wall, where it was deposited through chutes; in the drifts, under compressed air, the cost was about $10.70 per cu. yd.

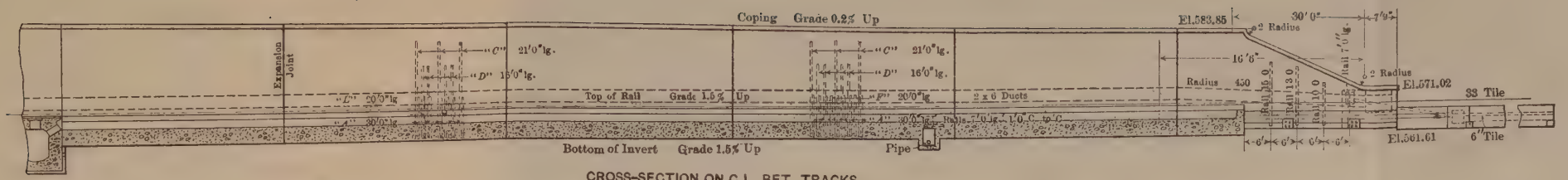
The work of water-proofing and timbering was similar to that for the Detroit approach.

Easterly Open Cut.

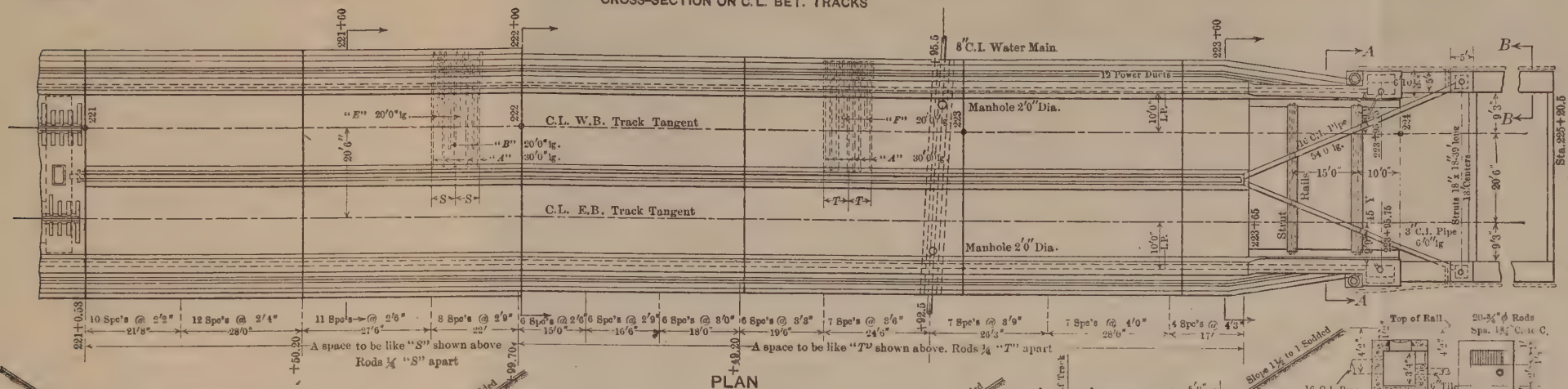
The easterly open cut extends from Station 220 + 58 to Station 250 + 00. The gradient, as noted, is 1.5%, and the maximum depth of cut at the portal of the tunnel is about 45 ft. From the portal, east to Station 224 + 00, the slopes are retained by walls which have a maximum height of 25 ft. at the portal. The tops of the walls are very nearly level, and are about 15 ft. below the original ground surface at the east end. The height and extent of the walls were governed by the width of the right of way north of the cut, on which the present main tracks are located. As will be noted by the location map, the tunnel tangent is not parallel to the northerly right-of-way line, and is nearest to it at the portal. The 1½:1 slope was calculated for the sides of the cut, and on this basis it was found necessary, in order to provide sufficient width of right of way for two tracks at the narrowest point, to carry the walls to Station 224 + 00. A cross-section of the walls and cut near the portal is shown on Plate XLI. Views of the finished open cuts at the portals are shown on Plate XLII.

As stated elsewhere, the building of the open cut destroyed the Railroad Company's large yard at Windsor. Additional property was

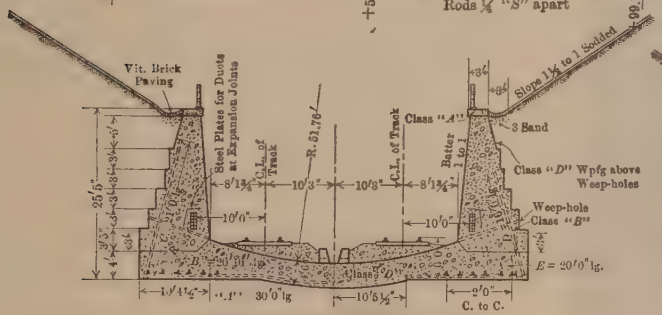
PLATE XLI.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXIV, No. 1212.
KINNEAR ON
THE DETROIT RIVER TUNNEL



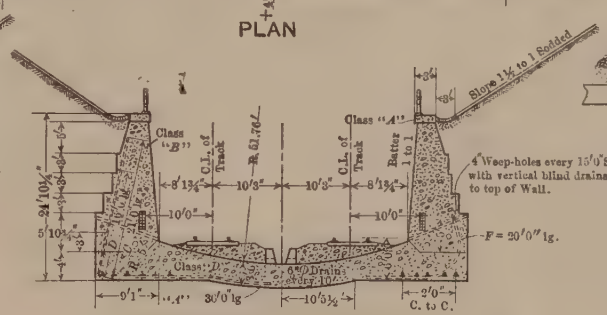
CROSS-SECTION ON C.L. BET. TRACKS



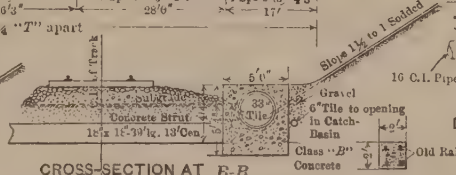
PLAN



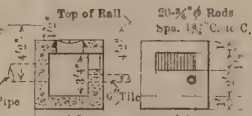
CROSS-SECTION AT STA. 221+60



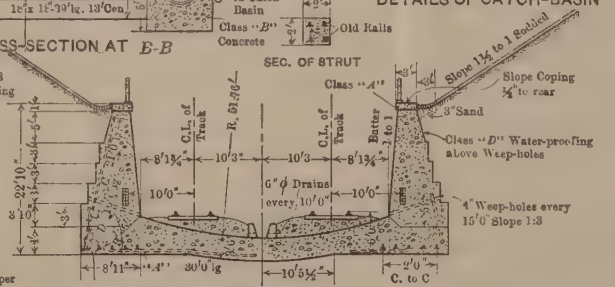
CROSS-SECTION AT STA. 222+00



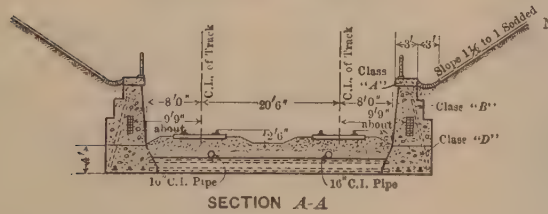
CROSS-SECTION AT *B-B*



DETAILS OF CATCH-BASIN

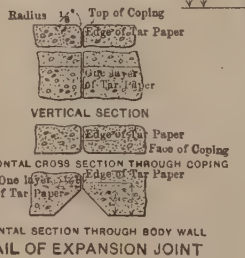


CROSS-SECTION AT STA. 223+60



SECTION *A-A*

Notes:
All exposed edges and corners of concrete to be rounded to 1" Radius
Expansion Joints to be spaced 50' C. to C. with tar paper joints.
Water-proofing on back of wall to be (Class "D") one coat of straight run coal-tar pitch ⅛" thick above Weep-holes
Track Rail 6" high 100 # per yd.
Rails in bottom of walls to break joints and to be lapped 3'0"
All Stations refer to W. B. Track.
All reinforcing rods 1" dia. round rods except 30' lengths which are ⅜" twisted rods
Back-fill for 18" back of wall to consist of cinders or other porous material from top of wall to Weep-holes



DETAIL OF EXPANSION JOINT

DETROIT RIVER TUNNEL
DETAILS OF EASTERLY OPEN CUT

acquired immediately, and a modern yard, complete in every detail, is now in operation.

Excavation.—The total excavation for the open cut, which was made by a steam shovel, was 240 000 cu. yd. The slopes were finished by hand, and sodded.

Concrete.—The walls are of concrete, of the same proportions and placed in the same manner as for the westerly open cut. The general design of the walls and details is also the same as for the westerly open cut.

The average cost for the 1 : 3 : 6 concrete in the retaining walls for the easterly and westerly open cuts was about $4.75 per cu. yd. The average cost for excavation in the easterly open cut was 32 cents per cu. yd., including the cost of the plant and incidental expenses, but not including the cost of disposing of the material, beyond loading it on cars and delivering them at the top of the grade, from which point they were handled by the Tunnel Company. The excavation in the westerly open cut cost about $1.40 per cu. yd., the higher cost there being due to more difficult working conditions, owing to its proximity to tracks and structures.

Drainage.—Provision has been made for taking care of most of the surface-water that will naturally drain into the cut, and of the drainage from the subway at Tecumseh Road, east of Windsor, through a sewer, the inlet of which is just below the bottom of the cut at Station 236 + 00, with an outlet at the river. This sewer was built by the Tunnel Company, and is 24 in. in diameter from the catch-basin to a point over the portal of the tunnel, where it increases to 36 in. Blind drains are provided at the back of the walls. The general drainage system for the tunnel is described later.

Subaqueous Tunnel.

The subaqueous section of the tunnel, as previously noted, is 2 668 ft. long, extending from Station 158 + 75 to Station 185 + 43. It will be seen by the plans that about 1 000 ft. of this section is level; that the vertical curves connecting the level with the grades of the approaches fall within this section almost entirely; and that both the easterly and westerly ends are on curves. The methods used in constructing this section were entirely new, as applied to subaqueous tunnels, and are particularly interesting, in view of the general success attained.

Design.—The plan, as adopted, consisted in dredging a trench of the required depth and width; sinking to correct position in the trench steel tubes in pairs, constructed so as to be water-tight; and surrounding the tubes with concrete, in plastic form, deposited in the water.

The surrounding, or exterior, concrete thus deposited, formed an artificial rock through which two water-proofed openings had been provided, and the inner lining, or tunnel proper, was built in normal atmosphere. The tubes, which were fitted at regular intervals with heavy wooden bulkheads and designed to stand the full hydrostatic pressure, were unwatered later, and lined with reinforced concrete, 20 in. thick. After the exterior concrete had been placed and before the tubes were unwatered, the remaining section of the trench on either side of the tunnel was back-filled with gravel and clay to the full height of the exterior concrete deposit.

The tubes are 23 ft. 4 in. in diameter, built of $\frac{3}{8}$-in. steel plates, riveted together and ship-caulked. They were sunk in pairs, 26 ft. 4 in. from center to center, each for a single track, and in sections 262 ft. 6 in. long, with two exceptions, one section, containing the drainage sumps, being 238 ft. 6 in., and the closing section, connecting with the Windsor approach tunnel, 64 ft. 6 in. The tubes were reinforced with 4 by 3 by $\frac{3}{8}$-in. circumferential stiffener angles riveted to the inside of the plate at intervals of 12 ft. The 3-in. leg was riveted to the shell. To strengthen the tubes further during launching, sinking, and concreting, twelve steel rods, $\frac{3}{4}$-in. square, were placed at each stiffener angle, radiating from a cast-steel ring at the center of the tube to the angle. These rods were bolted to the angle at points corresponding to the hour marks of a clock, passed through corresponding holes in the ring, and were made rigid by turning up a nut on each rod inside the ring.

For convenience in depositing the exterior concrete, the tubes were fitted at intervals of 12 ft. with steel diaphragms, $\frac{1}{4}$-in. thick, to the upright edges of which was attached heavy wooden sheathing. A cross-section of the subaqueous tunnel is shown on Plate XLIII. The sheathing served to retain the freshly deposited concrete within defined limits, and, with the diaphragms, formed pockets, each of which could be filled from top to bottom, independently of the adjoining pockets, in from 6 to 8 hours, thus providing for practically a monolithic structure throughout. The sheathing consists of planking, 6 in. thick at

FIG. 1.—WESTERLY OPEN CUT, DETROIT RIVER TUNNEL.

FIG. 2.—EASTERLY OPEN CUT, DETROIT RIVER TUNNEL.

the bottom of the diaphragm, diminishing to 3 in. at the top. It will be noted that the minimum thickness of the concrete outside the tubes is 3 ft. An 18-in. layer of gravel is shown below the bottom of the diaphragm, but for this was substituted a 1:4:8 mixture of concrete. The bottoms of the diaphragms are straight, thus giving the structure a level bearing on the foundation before the concrete was carried above the bottom of the diaphragms. The diaphragms on four sections of tubes are wider at the bottom than on the other sections, thus providing a greater bearing area, which was deemed advisable owing to the fact that the clay was slightly softer than that underlying the preceding and subsequent sections.

It has been stated that the entire tunnel was built through blue clay. Under the river the clay overlies limestone, which is about 90 ft. below the mean elevation of the water surface. The depth of trench varied from 60 to 80 ft. below the mean surface, and the average depth of the river bed along the line of the tunnel is 36 ft., the minimum being 18 and the maximum 48 ft. For a distance of several hundred feet near the middle of the river, the top of the tunnel is from 3 to 7 ft. above the bed of the river. Where the depth of the main river channel was originally 40 ft. deep or more, both the United States and Canadian Governments required that a depth of 40 ft. be maintained. The tunnel was built, leaving a depth of approximately 41 ft. in the main channel. Under the remainder of the river, the top of the tunnel is below the original bed of the river.

Work on this section of the tunnel was carried on at the same time as that on the approaches. The operations were conducted continuously from the point of connection with the Detroit approach to a connection with the Windsor approach.

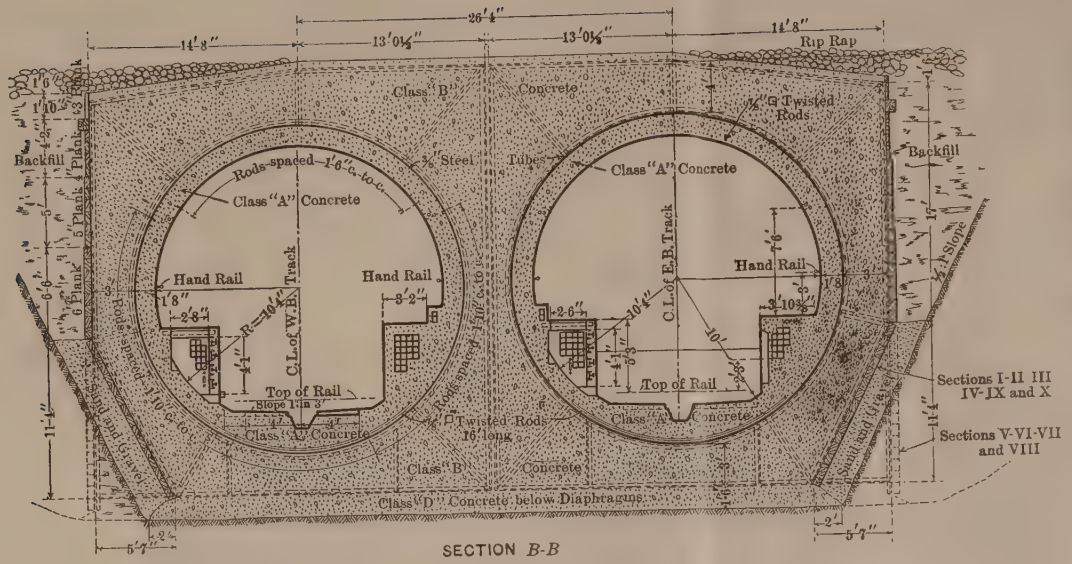
Excavation.—The greater part of the excavation for the trench was made with a clam-shell bucket, having a capacity of 3 cu. yd. The bucket was swung by a steel cable from a 60-ft. steel boom, and was operated by a double-drum hoisting engine. The plant was installed on a large scow equipped with two 24 by 24-in. steel spuds, 86 ft. long, placed aft for anchoring. This dredge removed an average of 600 cu. yd. of clay per day of 10 hours; the maximum quantity taken out in one day was 1 400 cu. yd., and the maximum for one month was 25 000 cu. yd., place measurement. The clam-shell is shown in Fig. 1, Plate XLIV.

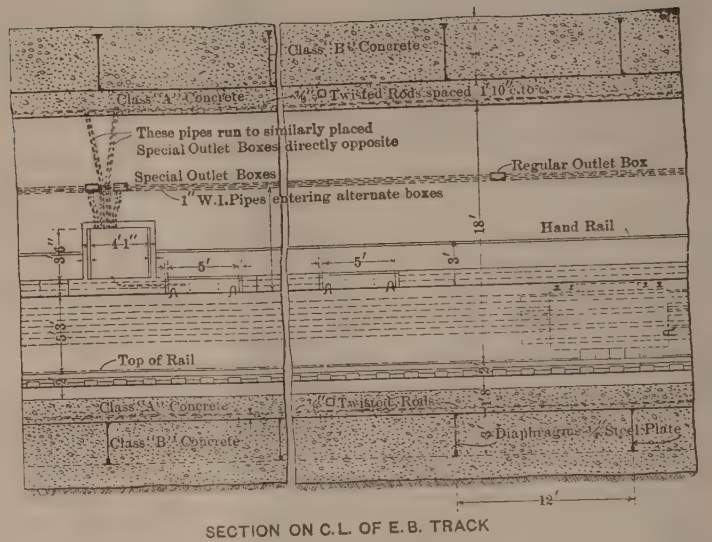
It was thought at first that the work might be done by a dipper dredge with a specially designed boom. This was tried, but without success. An ordinary dipper dredge was provided with an 80-ft. steel boom and especially large engines. About two-thirds of the first three sections of trench were excavated with this dredge, but to a depth of only 45 ft. At greater depths the strain on the anchoring spuds was excessive, causing very expensive breakages. After several months' work, the dipper dredge was replaced by the clam-shell, which operated successfully throughout the remainder of the work. This dredge would undoubtedly work equally well in 100 ft. of water.

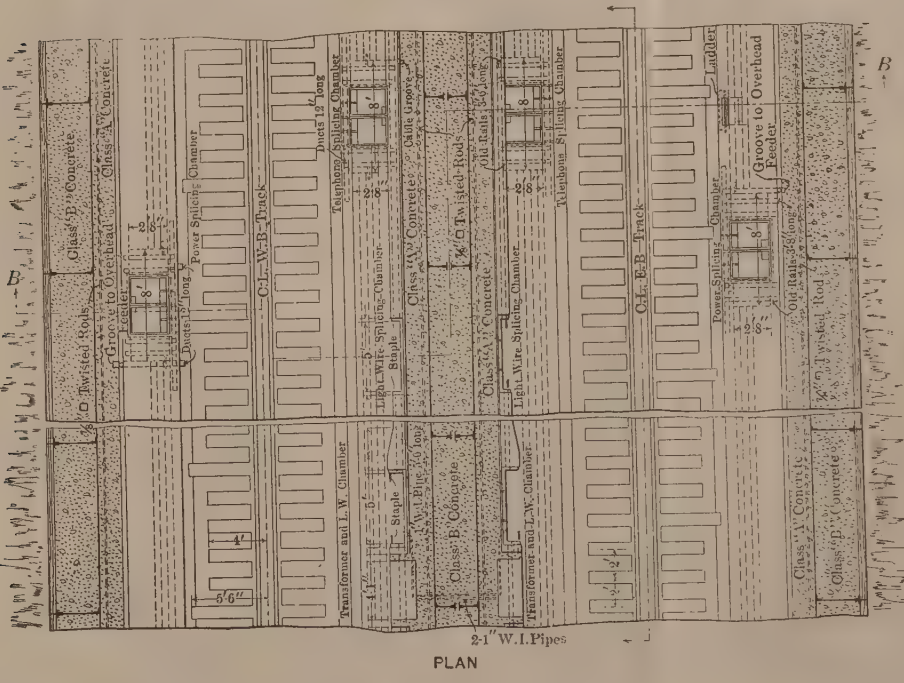
The clay from the trench was used as back-filling along the line of the tunnel outside the sheathing, after two sections of tubes had been placed and concreted. Prior to the sinking of the first two sections of tubes, all excavated material was towed in scows a distance of 6 miles down the river from the tunnel, and dumped at a point inshore from the harbor line. It might be interesting to note that great care had to be exercised not to waste any clay taken from Canadian waters across the boundary line into United States waters, unless for use as back-filling along the line of the tunnel. A violation of this restriction would undoubtedly have resulted in an imposed Customs duty.

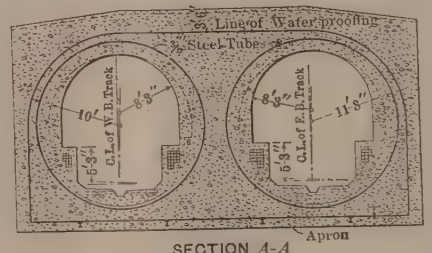
Sinking the Tubes.—In advance of the sinking of a section of tubes, and before placing the grillage for the temporary end support, it was necessary to determine the condition of the trench as to depth and width. This was accomplished with a 24-in. I-beam, 48 ft. long, suspended by cables under a derrick scow, and dragged along the bottom of the trench. The line of the sweep was regulated by noting the position of the suspending cables with relation to ranges on shore. An encroachment of any consequence would retard the sweep and was readily detected. As a further verification of the record, two divers followed the sweep and reported general conditions, such as depth of trench below the sweep and nearness of the slopes of the cut to the ends of the sweep. This latter information was interesting, but not absolutely necessary for the determination of clearances in the trench.

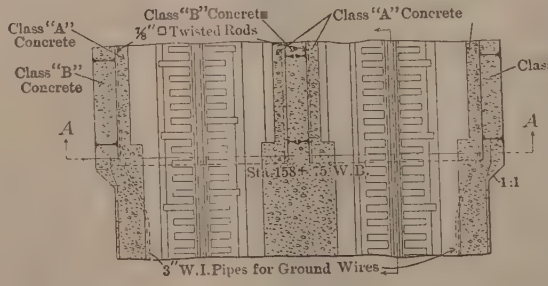
At intervals, marking the joints between the different sections of tubes, a steel grillage of I-beams was placed at the required level, under which concrete was deposited until the beams were engaged by the concrete. These grillages formed temporary supports for the ends of the sections, until the foundation concrete could be deposited. A




Class "B" Concrete
Class "A" Concrete
⅞" □ Twisted Rods spaced 1'10" c. to c.
These pipes run to similarly placed Special Outlet Boxes directly opposite
Special Outlet Boxes
Regular Outlet Box
1" W.I. Pipes entering alternate boxes
Hand Rail
Top of Rail
Class "A" Concrete
Class "B" Concrete
⅞" □ Twisted Rod
Diaphragm
Steel Plate
SECTION ON C.L. OF E.B. TRACK







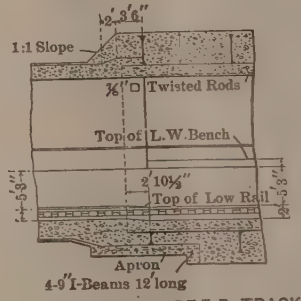


TABLE OF CONCRETE PROPORTIONS

Class "A"	= 1	Cement,	2	Sand,	4 Broken Stone.
" "B"	= 1	"	3	"	6 " "
" "D"	= 1	"	4	"	7½ " "

DETROIT RIVER TUNNEL

DETAILS, SUBAQUEOUS TUNNEL

plan of the grillage is shown by Fig. 7, Plate XXXIX, and a view is given in Fig. 1, Plate XLV. Each grillage was placed by suspending it on steel cables, handled by derricks on the large scow which formed part of the floating equipment. At each of the four corners of the grillage was attached a 10 by 10-in. timber leg, or spud, about 10 ft. long, which was driven into the clay by an ordinary pile-driver drop-hammer, swung from one of the derricks, until the grillage was brought to grade. The grade was determined with a steel mast, 80 ft. long, which was used as a level rod, the leveling instrument being stationed on shore. The mast was handled by a derrick, and the lower end was placed on the corners of the grillage by a diver.

For sinking, the ends of the tubes were fitted with water-tight bulkheads. Each of these bulkheads was equipped with a 14-in. gate-valve, which could be opened from the outside. Each valve was just below the water-line when the section of tubes was floating empty. The sections were sunk by opening these valves and permitting the tubes to fill with water. The bulkheads were fitted at the top with a 2-in. air-escape valve, to which was connected a 2-in. hose of sufficient length to permit the loose end to extend above the surface of the water when the section of tubes was in its final position. To the end of the hose above water was attached a 2-in. valve for controlling the escape of air. The tubes were equipped on the inside with two semi-bulkheads, placed 48 ft. from the end bulkheads. The semi-bulkheads for the first section had a maximum depth of 7 ft., for the others they were made 9 ft. deep, and when engaged by the water inside the tubes served to form, in connection with the end bulkheads, an air-chamber regulating the sinking of the section as a whole on even keel. In this manner the section was held in check and controlled in sinking until completely submerged. This control was established by valve-regulated air-escape pipes through the end bulkheads. When filling with water, the air in the tubes was permitted to escape through two 4-in. pipes, one back of each semi-bulkhead toward the middle of the tubes. These air-escape pipes were capped by divers after the tubes were in position in the bottom of the trench. The time required for submerging a section of tubes was approximately 2 hours.

Four air cylinders, or buoys, were used for floating the tubes to correct position, two being placed over each tube and connected to the diaphragms. The center compartment of these cylinders was of

$\frac{5}{16}$-in. steel; the others were of $\frac{1}{4}$-in. steel, and were 10 ft. 2 in. in diameter and 60 ft. long. Each cylinder was divided into three compartments; the length of the middle compartment was 13 ft., and that of the end ones was 23 ft. 6 in. Each compartment was fitted at the top with a valve for admitting water, also with an air-escape valve. The middle compartment was also equipped with a valve at the bottom, which was controlled from the top of the cylinder and used for the rapid admission of water, this compartment only being used in regulating the sinking of the tubes. The air cylinders were proportioned so that, with the end compartments empty, the tubes filled with water and were completely submerged, the entire section being held in suspension with about $1\frac{1}{2}$ ft. of the top of each of the cylinders visible above the surface of the water. The middle compartment was then permitted to fill with water, and the whole mass was lowered to its final position in the bottom of the trench. Further and more refined control of the section was obtained with two 5-ton counterweights, built of I-beams and handled by derricks mounted on a scow alongside. One counterweight was permitted to rest on top of each pair of air cylinders, and with sufficient water in the center compartment to overcome buoyancy, the additional load thus imposed produced a downward movement, which could be arrested at will. The cylinders were placed on the section of tubes, just before sinking, with a large A-frame on one of the scows, which was built as part of the plant to be used in connection with the sinking of the tubes. Fig. 2, Plate XLIV, is a view of this scow with the A-frame placing an air cylinder on one of the sections. After a section of the tubes was in proper position in the bottom of the trench and sufficiently anchored by concrete, the air cylinders were detached by a diver, brought to the surface, and held in readiness for use on the next section. One set of cylinders was used for all the sections.

The total weight of the material in a section of tubes 262 ft. 6 in. long, with four air cylinders, was 900 tons, the metal weighing 550 tons, the wooden sheathing and bulkheads 240 tons, and the air cylinders 110 tons. The weight of the mass in water, when entirely submerged, was approximately 516 tons.

For holding the tubes in position against the current during sinking, two concrete anchors, weighing in water 22 tons each, were buried in the bottom of the river 700 ft. up stream from the line of the tunnel

FIG. 1.—CLAM-SHELL DREDGE USED ON DETROIT RIVER TUNNEL.

FIG. 2.—AIR CYLINDER BEING PLACED ON SECTION OF TUBES.

and opposite the ends of the section which was being lowered. The large derrick scow, 35 by 120 ft., equipped with large hoisting engines of the Lidgerwood type, in addition to the two derricks mentioned, was anchored up stream from the permanent location of the section, and close enough for the derricks to handle the counterweights on the air cylinders. On the up-stream side of the tunnel section, attached to the diaphragms, were four large shackles, two at each end, one at the top and one at the bottom. To each of these four shackles was attached a set of tackles, consisting of 2-in. manila line, run through two sets of quadruple blocks. One of the blocks was attached to the shackle and the other to a 1⅜-in. cable leading to one of the anchors. The lines from the tackles were run to the drums of the engines on the scow, from which the whole system was controlled. Fig. 1, Plate XLVI, is a view of a section of tubes being sunk. Fig. 4, Plate XXXIX, is a sketch of the concrete anchor.

As has been stated, operations on the river were conducted from Detroit, toward Windsor, so that the west end of each section of tubes following Section No. 1 was connected to the east end of the preceding section. The connection was made by a specially designed joint, Fig. 5, Plate XXXIX. On top of each tube at the west end was attached a pilot pin, so-called, 6 ft. long and 6 in. in diameter, slightly tapered at the end, which fitted into a socket, 6½ in. in diameter, on the east end of the preceding section. The pin near the end was slotted to receive a steel wedge, or key. This key, when driven home and locked, fitted tightly against the socket casting, thus holding securely in position the west end of the section of tubes which was being placed. With the pilot pins in place in the sockets, a tight joint was made between the connecting sections with the abutting ends of the tubes coinciding. The special joint provided an exterior annular pocket enclosing the connection, 4 in. deep and 8 in. long, which was made practically water-tight by an arrangement of rubber gaskets. After the tube was unwatered, this pocket was filled from the inside with neat cement grout. In addition to the pilot pins, the connection was further strengthened by bolts on the outside, which were placed by divers. To make the joint in the manner described, it was, of course, necessary that the east end of the section being placed should be in its final position.

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The position of the section with respect to line and grade was determined by steel masts, one on each tube at the east end and one on the west-bound tube at the west end. These masts consisted of two channel-beams, 10 in. wide, set back to back, 10 in. apart, and connected by lattice bars. On each face was marked the center line, and the mast was graduated in feet, the numbering being referred to the center of the tube. Each mast was carefully set with its center line at right angles to the horizontal diameter of the tube and on center. Lines and grades were given by instruments on shore, and the tubes were adjusted by manipulating the holding lines.

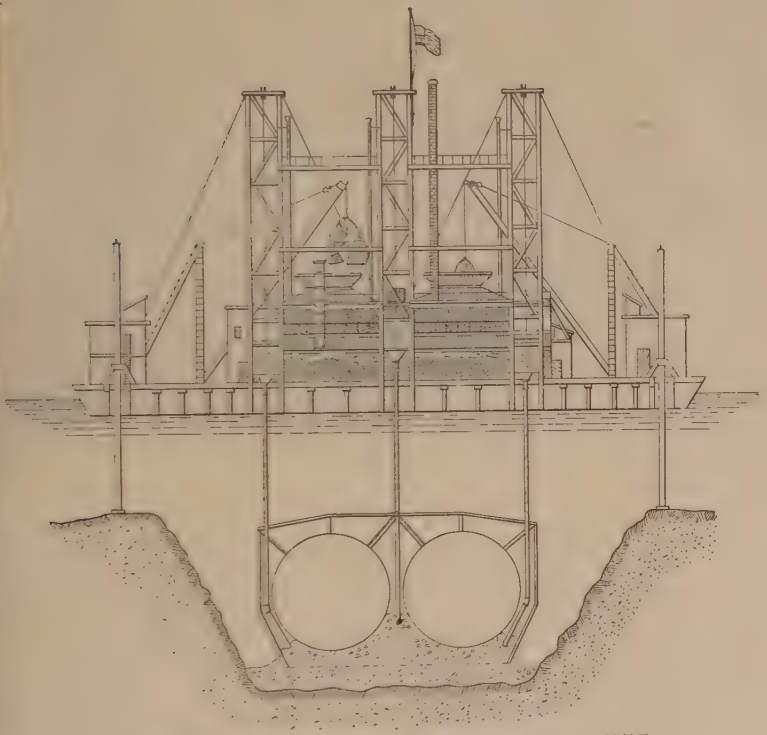
The grillage was usually set several inches below the established grade of the bottoms of the diaphragms, in order to guard against its becoming too high, and shims were inserted between the bottom of the diaphragm and the beams of the grillage. This work was done by divers. The grillage was designed to engage four diaphragms, the two end ones of each section, but shims were placed under the end diaphragm only. It will be remembered that the tubes were carried almost in suspension by the air cylinders until the section was anchored by the exterior concrete deposit. When a section was finally adjusted, it was anchored temporarily to the grillage by a turnbuckle arrangement, Fig. 8, Plate XXXIX, which was placed and made secure by divers. A remarkable feature in connection with the adjusting of the sections to final position was the perfect control which was obtained at all times, resulting from the use of the air cylinders. It was possible to raise or lower the tubes a fraction of an inch with the greatest ease and precision by regulating the quantity of water in the cylinders. The time consumed in sinking a section of tubes, adjusting, bolting, and making ready for the concrete, ranged from 3 to 7 days.

Exterior Concrete.—For depositing concrete, an especially large and well-equipped scow was built by the contractor. It was 36 by 155 ft., and was fitted with 20 by 20-in. timber spuds 90 ft. long. It carried a two-story structure which housed the concrete mixers, hoisting engines, and dynamo for lighting purposes, all of which were on the lower deck or floor of the scow. In addition to these there were two large derricks on the lower deck, which, when concrete was being deposited, were used for transferring gravel from the barges to the scows, gravel having been used on this work exclusively. The upper story of the superstructure contained three screening hoppers, above

FIG. 1.—STEEL GRILLAGE USED AS TEMPORARY SUPPORT FOR STEEL TUBES.

FIG. 2.—TREMIE SCOW IN OPERATION, SUBAQUEOUS SECTION, DETROIT RIVER TUNNEL.

ch were two bins, each with a capacity of 75 cu. yd., for receiving gravel. The gravel was drawn from these bins and screened into three hoppers. On the floor of the second deck, immediately under screening hoppers, were three small hoppers just above the mixers, proportioning the materials. On one side the scow was provided th three leads, each carrying a tremie pipe. These pipes were



METHOD OF PLACING CONCRETE BY TREMIE.

FIG. 7.

26 ft. 4 in. from center to center, and extended to a height of 89 ft. above the surface of the water. These leads also carried the buckets for elevating the concrete to the upper ends of the pipes. At the top of each pipe there was a hopper for receiving the concrete, the connection of the hopper with the pipe being such that the pipe was free to swing laterally. The hopper carried a cage in which a man was stationed to direct the handling of the pipe when concrete was being

deposited. The pipes were 12 in. in diameter, of spiral-riveted wrought iron, in 16-ft. lengths, with flange connections. The buckets for elevating the concrete were designed to dump automatically into the hoppers at the tops of the tremies. Fig. 2, Plate XLV, shows the scow and the tremies in operation. Fig. 7 is a sketch of the scow.

With a section of tubes in place, ready for depositing the exterior concrete, the scow was floated to position, the tremie pipes being placed directly over one of the pockets formed by the diaphragms where it was desired to work. The tremies were then lowered until the bottom ends were about 1 ft. above the bottom of the trench, with the middle tremie in the center between the two tubes and the side tremies between the outside of the tubes and the sheathing. To avoid delay, the tremie pipes were generally located in the pockets by divers. The pipes were left open at the bottom and filled with concrete to the exclusion of the water. To accomplish this the first two or three batches of concrete were mixed very dry, and the first two batches seldom failed to be effective. After the lower end of the pipe was sealed against the inflow of water by the initial deposit of the dry mixture, the concrete was mixed wet, as compared with the usual consistency of that placed in air, thus admitting of easy flow through the pipe. Under good working conditions, the rate of deposit through one of the tremies averaged about 30 batches per hour. The lower end of the pipe was generally held from 3 to 5 ft. in the concrete, and the concrete was discharged by raising the pipe slightly as each batch was dumped in the hopper at the upper end. The flow was carefully regulated, so that the pipe was filled with concrete from the beginning to the end of a run.

Due to the absence of any reliable data or records of experience in depositing large quantities of concrete in water of such depth as was found on this work, the contractors conducted numerous experiments with the tremies in the open trench before the actual work was started. Some very valuable information was gained through these experiments, resulting in the continuously successful operation of the tremies. In the early experiments it was found very troublesome to secure a slow discharge of the concrete. At times the pipe would be raised 8 or 10 ft. for the purpose of starting the flow, which would usually result in a rapid discharge of the concrete and the pipe filling with water. This action was found to be caused by the bottom of the pipe becoming clogged with clay when first lowered into the trench. The wad of clay

FIG. 1.—SECTION OF TUBES DURING SINKING.

FIG. 2.—SECTION OF TUBES IN SUSPENSION BY AIR CYLINDERS.

was dislodged only after considerable jarring of the pipe, by raising and lowering it, and the charge of concrete was usually lost in the operation. This difficulty was the only one of importance encountered in depositing the concrete with the tremies, and, of course, was readily overcome after the cause was discovered. It was found, however, that the best results could be obtained only by observing the utmost care in the manipulation of the pipes.

Each side tremie, when depositing in the sections with sloping sides, was provided with a knuckle-joint near the lower end, so that the lower section of the pipe was free to swing laterally and was thus directed under the tube. This lower section was of sufficient length to reach the bottom of the trench. When the concrete had been carried up to the top of the sloping side, the lower section of pipe, together with the knuckle, was removed.

The usual order of procedure, in concreting under and around the tubes, was to fill one of the pockets near the easterly end of a section as soon as possible after the section was in position, thus forming a positive anchorage against lateral movement due to the action of the current in the river. After this pocket was filled, foundation concrete was deposited in the bottom of the trench under the entire length and width of the section. The concrete for the foundation, up to the bottoms of the diaphragms, consisted of one part cement, four parts of sand, and eight parts of gravel. The concrete above the bottoms of the diaphragms was a 1:3:6 mixture. The foundation concrete, deposited as it was under the entire section directly after the tubes were placed, sealed the bottom of the section against an inflow of clay from possible slides along the slopes of the cut and from any other foreign matter which might be carried into the trench. It also gave a bearing to the tubes over their entire length.

After the foundation concrete had been deposited, a pocket near the middle of the section was filled, thus securing an intermediate bearing. It will be remembered that the air cylinders were not removed until the tubes had been anchored and given a bearing as described. After the removal of the cylinders, the remaining pockets in the section were filled, the work being carried on continuously, as nearly as possible, from end to end of the section. Concreting operations in any one pocket were carried on continuously, or as nearly so as possible, from bottom to top, thus forming a monolithic block the size of the pocket. It will

be noted that the plans show the concrete 6 in. above the top of the diaphragm, or 4 ft. 6 in. above the top of the tube. No attempt was made to finish off the top of the concrete; it was not disturbed when once deposited. The placing of the concrete was inspected by a diver, whose duty it was to see that it was carried to at least 6 in. above the top of the diaphragm at all points. The elevation of the concrete was afterward verified by soundings. The depth of the foundation concrete under the entire tunnel varies from 1 to 6 ft., according to the depth of trench left by the dredge.

The total quantity of concrete deposited under and around the tubes was 101 900 cu. yd., and it was placed in a working period of 18 months. The maximum quantity placed in one day of 16 hours was 1 069 cu. yd., and the maximum for any one month was 10 287 cu. yd. The average number of yards per day deposited around the tubes above the foundation, for the days on which work was done, was 349. The average cost of the 1:3:6 concrete was about $4 per cu. yd., and the average cost of the 1:4:8 foundation concrete was about $3.75. These figures include all costs for material, labor, and equipment.

It may be interesting to know something of the character of the 1:3:6 tremie-deposited concrete, as developed on this work. A large number of cores, about 5 in. in diameter, were taken from one of the pockets by a 6-in. Davis calyx shot drill, operated from the surface of the river. These cores were tested in compression and developed a strength as high as 4 000 lb. per sq. in., the lowest being 2 740 lb., from a series of six cores chosen at random. The specimens were broken one year from the date of deposit. At one point borings were made through a depth of 35 ft. of concrete, from which 92% of the cores were obtained, ranging in length from 6 to 20 in. The remarkable character of the tremie-deposited concrete was later disclosed in two concrete bulkheads, formed by depositing the concrete inside of the tubes between the wooden bulkheads in two adjoining sections. The tremie pipe entered the tubes through a manhole (provided for other purposes) from which the cover was removed by a diver. These concrete bulkheads, by way of explanation, were placed as additional protection to the workings in the tubes previously sunk, until succeeding sections could be placed. It required the continuous use of explosives and three weeks' constant labor to remove each of these bulkheads. This concrete was a 1:2:4 mixture, and was particularly dense, as were all the other

samples of the subaqueous concrete. Some 6-in. cubes, cut from the bulkhead concrete, developed a strength of from 1 800 to 3 040 lb. per sq. in. in compression. Probably the strength of these specimens was somewhat impaired by the work involved in shaping.

At two special joints the tubes were surrounded by a 1:2:4 mixture of concrete, and, after the tubes were unwatered, it was found that these joints were nearly water-tight. This is a development which must be considered in connection with the hydrostatic pressure, found to be about 30 lb. per sq. in. at the bottom of the tubes.

Back-Fill.—The work of back-filling the trench outside the sheathing was started immediately after two sections of tubes had been lowered and concreted. For the most part gravel was used to a height of 11 ft. above the foundation concrete, and the remainder of the trench was filled with clay as excavated by the dredge. Where the top of the tunnel projects above the original river bottom, the back-filling was carried up to the level of the concrete. Later, the clay back-fill was covered with rip-rap, deposited to a depth of about 2 ft., as a protection against the possible scouring action of the current. Where gravel was used as back-filling, it was deposited through 12-in., wrought-iron pipes, lowered into the trench from the gravel barges. The clay was dumped directly from the drop-bottom scows, in which it was loaded by the dredge, to a height determined from time to time by soundings. The rip-rap was dropped by a clam-shell from the surface of the water, the position of the scow carrying it being determined by ranges on shore.

Unwatering the Tubes.—The west end of the first section of tubes and the east ends of Sections Nos. 1, 3, 5, 7, 9, and 10, were each provided with a heavy bulkhead, of 10 by 12-in. yellow pine timbers, heavily braced, and of sufficient strength to withstand the full hydrostatic pressure when the tubes were unwatered. There was also a heavy bulkhead at the east end of Section No. 2. The east ends of the other sections and the west ends of all sections, except Section No. 1, were provided with 4-in. bulkheads, useful only in connection with the sinking. In addition to the 14-in. inlet-valve, already referred to in connection with the sinking as being located outside the bulkhead, each bulkhead was fitted on the inside with a 10-in. valve for unwatering. It was found expedient during the progress of the work to use the 10-in. valves in the heavy bulkheads only, leaving the 14-in. valves

on all the light ones open at all times, thus giving a continuous opening through two sections of tubes between two heavy bulkheads. The 14-in. valves on all heavy bulkheads were closed by the divers directly after the tubes were sunk. With this arrangement it was possible to unwater Section No. 1 before drawing the water from Sections Nos. 2 and 3, and likewise Sections Nos. 2 and 3 were unwatered through Section No. 1 without opening Sections Nos. 4 and 5, and so on across the river. In this manner it was possible to begin the placing of the interior lining of concrete before all the sections had been sunk.

The water was pumped from the first five sections of tubes by an 8-in. centrifugal pump, placed in the bottom of Section No. 1 in advance of its sinking. This pump was also used in unwatering all sections east of No. 5, being assisted by a similar pump placed at the east end of the latter. For carrying the water to the pump, a 12-in., wrought-iron pipe was laid through each of the first five sections in advance of sinking, that in Section No. 1 being connected with the pump. An access pipe, 4 ft. in diameter, designed to extend above the surface of the water with the tubes in place, was provided for each tube at the west end of Section No. 1, also a 12-in. discharge pipe. All these pipes were built as a part of the tubes before the section was lowered. The pumps were placed directly under the access pipes, and the pump shafts led up through these pipes.

After five sections of tubes were in position, and before the coffer-dam connection with the shore work was completed, Sections Nos. 1, 2, and 3 were unwatered, and preparations were made for placing the concrete lining. Sections Nos. 4 and 5 were next unwatered, and this work was continued progressively in pairs to the Canadian connection. Work on the concrete lining was started in Section No. 5, after seven sections of tubes had been placed and concreted outside, and before the water was drawn from Sections Nos. 6 and 7. This was directly after the coffer-dam connection was made between the Detroit shaft and the first section of tubes, and all material, except concrete, was handled through the shaft.

It may be of interest to note that when the different sections of tubes were unwatered, they were found to be water-tight, with the exception of a slight seepage at some of the rivets and at the lap-joints between the plates, where they may have been strained in launching. These small leakages were stopped by caulking with steel

caulking irons and by the application, in a few cases, of red lead forced into the lap-joints by a small pump commonly used for such work.

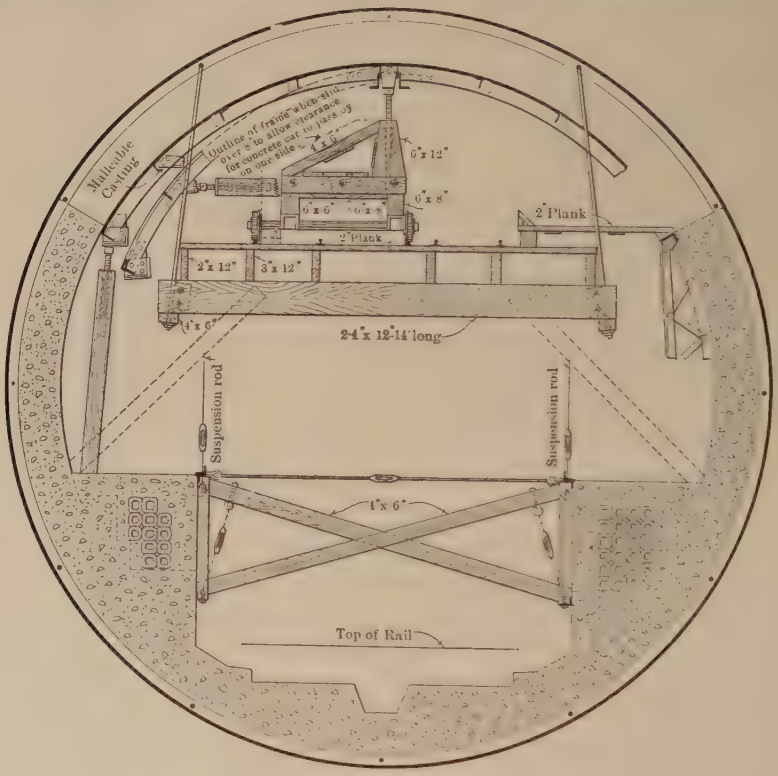
Concrete Lining.—The cross-section of the concrete lining in the subaqueous section is shown on Plate XLIII, which also shows the location and size of the reinforcing rods used throughout this section, and the invert reinforcement in the approach tunnels. The concrete for the full section, from invert to arch, except that over the ducts, was placed in four runs, namely, the invert, to 12 in. below the top of the bench-walls; the two side-walls, to 8 ft. above the top of the bench; and, finally, the arch. The ducts were laid and concreted after the inverts, side-walls, and arch had been placed throughout the tubes. All concrete for inverts, side-walls, and arches was delivered in the tunnel over a platform suspended from the top of the tube at about the level of the top of the side-walls. Fig. 8 shows the details of this platform. Steel forms were used for the concrete lining, 24 ft. long for inverts and side-walls, and 12 ft. for the arch. The placing of concrete for inverts and side-walls was simple, but the arch work required an end handling with shovels, followed by a ramming to place. All concrete for lining the subaqueous tunnel was mixed on the surface and delivered through chutes placed in the access pipes, previously mentioned. As the work of placing the lining progressed, the suspended platform was moved from the rear and placed in advance of the work as rapidly as the tubes were opened up, the narrow-gauge material tracks being shifted from the platform to the completed invert. Cars of material were handled from the low-level tracks to tracks on the swinging platform by an elevator. As all concrete for the tubes was mixed at the Detroit end, there was a haul of more than 2 700 ft. for the concrete placed near the Windsor connection. The cars were drawn by cables, and 30 min. was the longest time required for delivering a batch of concrete from the mixer to the farthest point of deposit. The concrete lining was completed in exactly 12 months from the time of starting. Interior views of the subaqueous tunnel are shown on Plate XLVII.

Connection Between the Subaqueous Tunnel and the Westerly Approach Tunnel.

Under the heading, "Westerly Approach Tunnel," it has been mentioned that the connection between the westerly approach and the

subaqueous sections was made in the dry by a coffer-dam. By the time this connection was completed, seven sections of the steel tubes had been sunk and concreted, and the first three sections had been unwatered.

Shortly after the completion of the Detroit shaft, the westerly approach arches were built, in drifts, 2 ft. east of the shaft, to Station



DETROIT RIVER TUNNEL
SUSPENDED FLOOR CONSTRUCTION
AND POSITION OF FORMS

Fig. 8.

158 + 23, leaving 52 ft. of tunnel section to be built in the open, the westerly end of the first section of tubes being at Station 158 + 75.

The original shore line on the Detroit side was 65 ft. east of the point where the connection was made between the subaqueous and westerly approach sections. The excavation for the trench to receive the first section of tubes thus left a small bay, about 200 ft. wide, at

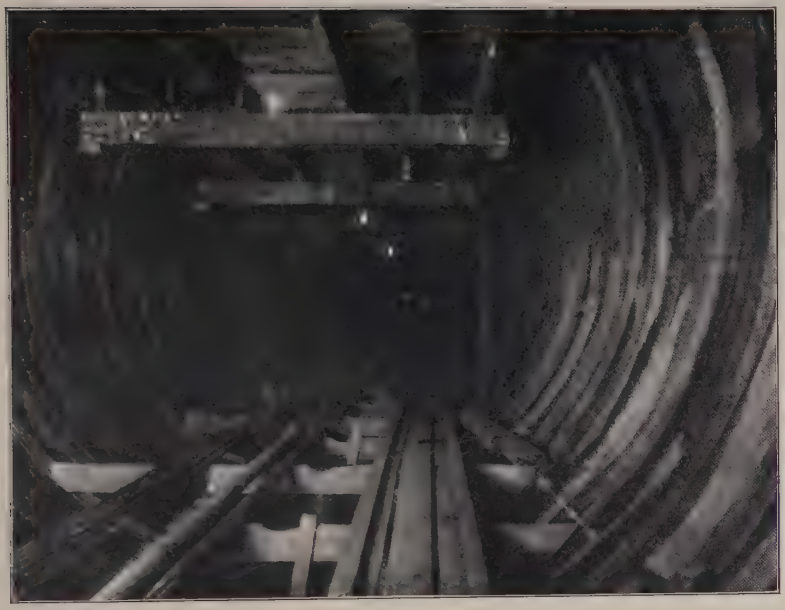


FIG. 1.—INTERIOR OF SUBAQUEOUS SECTION BEFORE CONCRETE LINING WAS PLACED.

FIG. 2.—INTERIOR OF COMPLETED SUBAQUEOUS SECTION.

the water surface, over the ends of the tubes. The depth of the trench below the water surface at this point was from 55 to 60 ft., and the width at the bottom was about 60 ft.—slightly wider than the section of tubes. The side slopes, therefore, averaged a trifle more than 1:1. The ground surface at this point was about 5 ft. above the surface of the water, and the top of the concrete over the shore end of the tubes was 25 ft. 6 in. below the surface of the water, the latter in every case being the mean elevation, 574.2.

The enclosure between the ends of the tubes and the shaft was made by building a dam across the bay over the end of the first section of tubes. This dam consisted of concrete with a clay back-filling, deposited between rows of heavy wooden sheet-piling, on either side of the tunnel, and a crib sunk over the end of the tunnel, together with a further back-filling of clay outside of the crib and piling. The crib extended 30 ft. beyond each side of the tunnel, over the deepest part of the trench, thus providing ample width for the concrete wall. Having an inside width of 23 ft., the rows of sheet-piles along the crib meeting the sides of the tunnel, were 26 ft. apart inside. The two rows of sheet-piles extending from each end of the crib to the bank on either side were 25 ft. apart at their connection with the crib, narrowing down to 10 ft. at the bank as the water became more shallow, and were driven with the outside row at an angle of about 30° with the connecting row along the crib to form, as it were, a wing-wall at either bank.

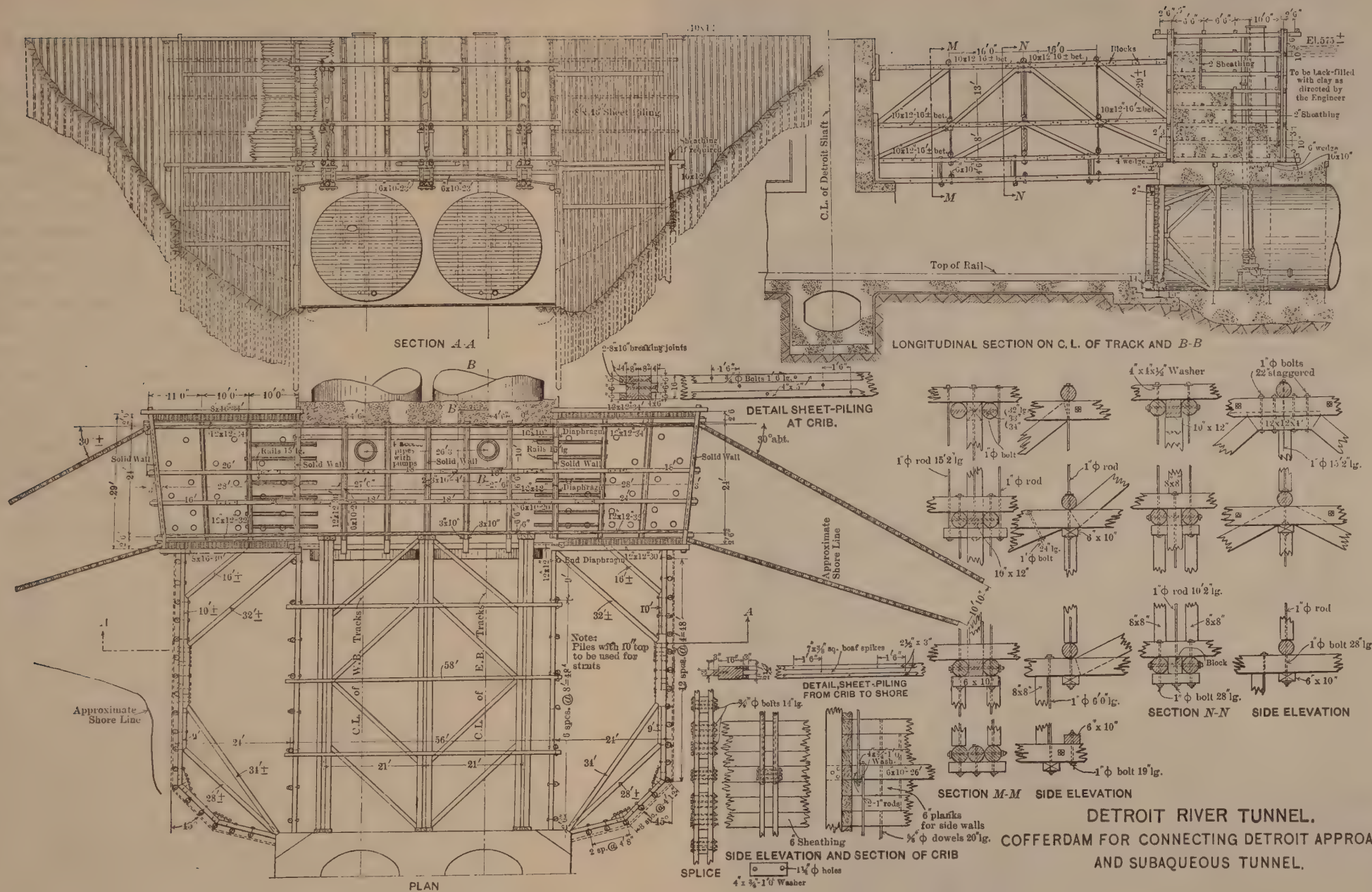
The crib, 111 by 23 ft., with a total height of 29 ft. 6 in., was assembled in the water over the tubes and sunk as timbers were added, being guided to position by the piles driven in the trench inside of the lines of its proposed location. When in position, the crib rested on 12 by 12-in. timbers, which had been attached to the tops of the two end diaphragms before the tubes were sunk. For convenience in concreting, the crib was divided into four compartments, separated by solid timber walls at right angles to its axis, one over each side of the tunnel sheathing and one over the center line of the tunnel. All the outer and division walls were of 6 by 12-in. timbers. After the crib was in place, the sheet-piling was driven. This consisted of 16 by 16-in. timbers along the crib, built up of 8 by 16-in. bridge stringers, and 12 by 12-in. pieces in the wings, built up of 6 by 12-in. car-sills. The length of the heavy 16 by 16-in. piles ranged from 50 to 80 ft., and the

others from 30 to 50 ft. The piling was driven to a penetration of from 12 to 15 ft. The 16 by 16-in. assembled timbers were connected by tongued and grooved joints by attaching to one side of each piece two 4 by 5-in. splines, making a 4 by 6 in. groove, and to the opposite side a 4 by 6-in. piece to serve as a tongue. The 12 by 12-in. piers were connected in the same manner.

Concrete was deposited first in the side pockets of the crib, up to the top, above the surface of the water, and considerable time was allowed to elapse between the filling of these pockets and those over the tunnel, so that any settlement of the concrete in the side pockets would not cause settlement of the tubes, as the concrete over the latter was bonded to that on the sides by steel rails. The space between the rows of piles under the side pockets of the crib, up to the top of the tunnel, was filled with concrete; above the level of the top of the tunnel, the concrete in the side pockets and over the tunnel was deposited to the shape of a gravity retaining wall, 6½ ft. at the top, stepping off on the river side to the full width of the crib at the top of the tunnel. The remainder of the space in the crib was filled with clay. The wings were filled with concrete to within 12 ft. of the surface of the water, and above this the space was filled with concrete to an average width of 10 ft. on the inside and with clay on the river side. To complete the dam, clay was deposited in the river, next to the sheet-piling and the crib, to the surface of the water.

On the completion of this heavy cut-off wall, which was 228 ft. long, with a maximum height of 60 ft., the water was pumped from the enclosure between the wall or dam and the tunnel shaft, and the necessary excavation was made, to complete the tunnel section from the shaft to the tubes. As the water was lowered and the excavation was carried down, heavy, horizontal timber braces were placed, reaching from the dam to the shaft, a distance of 55 ft. These braces were arranged in five courses from the level of the water in the river down to the top of the tunnel, and were braced horizontally and vertically to prevent deflection. The braces formed a mutual support between the dam and the shaft, although they were used more to prevent a listing of the shaft than as a support for the dam. The coffer-dam and bracing are shown on Plate XLVIII. As the excavation was carried downward, the sides of the cut were retained by rows of sheathing driven at several levels, forming a series of terraces. The sheathing

PLATE XLVIII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXIV, No. 1212.
KINNEAR ON
THE DETROIT RIVER TUNNEL.



was braced securely by cross-struts extending from side to side of the cut. There was evidence of tremendous pressures on the transverse struts, but none on the longitudinal braces between the dam and the shaft. The material was excavated by hand, and removed in buckets handled by derricks set up around the top of the work. The closing section of the tunnel was built without any trouble, as there was very little seepage through the dam, and the work was completed in a very short time after the excavation.

This section of the tunnel was water-proofed with ten layers of felt and eleven of pitch, applied alternately, as previously described. The concrete connecting with the tubes was placed so as to surround their ends completely, with a minimum thickness of 12 in., and was extended 2 ft. east of the ends of the tubes, to meet the first steel diaphragm. The water-proofing was carried to a lap-joint at the diaphragm, and this union was covered with a very rich mixture of concrete to a thickness of 3 ft. over the arches and 2 ft. on the sides down to the invert.

The space enclosed by the coffer-dam was back-filled with clay up to the original surface of the ground as soon as the connection was made and the water-proofing completed.

Connection Between the Subaqueous and the Easterly Approach Tunnels.

As mentioned in the description of the easterly approach tunnel, that portion of this section of the tunnel immediately adjacent to the subaqueous tunnel was completed under compressed air, in drifts, down to the actual point of connection. The extrados of the arches at the end of the concrete section outside of the water-proofing was covered by a steel plate, which was bent normal to the axis of the tunnel over the end of the section, and served as a water-proof surface against which the steel tubes abutted when in place, thus providing a water-tight joint similar to that between the sections of tubes, shown in Fig. 5, Plate XXXIX. When the trench was excavated for the connecting section of tubes, the clay was removed by the dredge from around the end of the approach tunnel, and the tubes were sunk to place in the usual manner. To prevent an inrush of water to the approach tunnel when the end was uncovered, three heavy concrete bulkheads, 50 ft. apart, provided with steel doors, were

built as a part of the tunnel close to the point of the proposed connection.

A grillage was sunk in the trench adjoining the approach tunnel, to provide a temporary support for the tubes. This grillage was similar in every respect to those used at other joints, except as to width. The closing section of tubes, known as Section No. 11, was 64 ft. 6 in. long, and the gap between the easterly end of Section No. 10, the last full-length section, and the approach tunnel was 65 ft. 4 in., thus leaving 10 in. between Sections Nos. 10 and 11 when the latter was in place against the end of the approach tunnel. The opening thus left between these sections afforded an opportunity to use a joint of special design, which was easily made by the divers, and the closure was sealed by the deposit of exterior concrete. When the tubes were unwatered, they were made continuous over this joint by inserting a plate on the inside, which was bolted to the tubes and caulked so as to be water-proof. The joint provided a small annular space between the plate inserted and the exterior concrete. This space was grouted from the inside of the tubes, thus eliminating any possibility of voids between the outside of the tubes and the exterior concrete.

Construction of the Steel Tubes.

The contract for the construction of the steel tubes was sublet by the general contractor to the Great Lakes Engineering Works, of Detroit. The work was done according to plans and specifications approved by the Detroit River Tunnel Company, and under the latter's supervision. The tubes were erected at the St. Clair plant of the Great Lakes Engineering Works, at St. Clair, Mich., 60 miles north of Detroit, on the St. Clair River.

The method of laying out and assembling the shell-plates was similar in every way to that used in the building of lake freighters. All plates were made from templates laid out in the mould loft from developed surfaces. This method is known, throughout the United States and Great Britain, in shipbuilding work, as the "Universal System of the Great Lakes."

The plates for the tubes were riveted together with lap-joints and attached to the steel diaphragm by an angle-iron, running entirely around the tube, riveted to the diaphragm and to the tube plate, the diaphragms being spaced so as to come at the center of the plate

between the lap-joints. The tubes and diaphragms were erected in the following manner: The diaphragm plates were assembled and laid out on wooden barrels; butt-straps were bolted to the butts of the plates, and the angle-bars which connect to the sheathing were bolted on; finally, the angle-irons to connect with the tube were bolted in place, thus giving a big frame of plates and angles in which was a hole 26 ft. 4 in. in diameter. Four half diaphragms were laid out at one time in the manner described, and then raised to position on keel-blocks, as shown on Plate XLIX. The keel-blocks rested on concreted beds, and were carefully leveled before erecting work was started. The diaphragms were held temporarily in an upright position by longitudinal tie-angles bolted across the tops. When the diaphragms were erected and properly supported, the curved shell-plates were bolted in place temporarily, the intermediate plates between the diaphragms being placed last in all cases. The plates were cut so that the centers of the vertical lap-joints were 6 ft. apart, four plates forming a complete ring 6 ft. long. They were assembled with the horizontal joints of the successive 6-ft. rings staggered.

When an entire section of tubes was erected and securely bolted, and before riveting was started, the tubes were checked for alignment and grade, and any necessary change was made by moving the diaphragms.

After correction was made for alignment and grade, the riveters were started at work, and following them the caulkers. Air tools were used in both riveting and caulking. The caulking was done just as in boiler construction, the plate edge at the lap being flattened cold and pressed, or annealed, so to speak, into the plate underneath.

To insure a good fit at the joint between the sections of tubes, when they should finally be in place in the bottom of the trench, the connecting ends of adjoining sections were fitted together in the shipyard and tested with a water pressure of about 30 lb. per sq. in., which corresponds closely with the hydrostatic pressure at the lowest point of the tunnel.

The wooden sheathing along the sides, for retaining the concrete deposit around the tubes, was placed at St. Clair, as were also the wooden bulkheads, before the tubes were launched. Very little work remained to be done on the different sections of tubes after their arrival at Detroit.

About 20 months were required for building all the sections of tubes complete.

Tunnel Details.

Clearance.—The tunnel as built provides an overhead clearance from top of rail to crown of arch of 18 ft. 0 in., the arch section being a semicircle with a radius of 10 ft. The distance between bench-walls, the tops of which are 5 ft. 3 in. above the top of rail, is 11 ft. 6 in.—5 ft. 9 in. on either side of the center line, on a tangent, and increasing $\frac{3}{4}$ in. per degree of curve, or $\frac{3}{8}$ in. on either side of the center line.

Safety Ladders.—For safety and convenience, ladders were provided throughout the tunnel, at intervals of 50 ft., in each bench-wall. They extend from the level of the top of rail to the top of the bench; the rungs are of 1-in. pipe, 2 ft. long, built into niches in the wall, 4 in. deep. In the top of the bench, just over the ladder, there is a countersunk hand-hold of 1-in. pipe. The ladders in the two walls of each tube are staggered so as to make the distance between them 25 ft.

Lighting.—The tunnel is lighted by 16-c.p., 110-volt, incandescent lamps, at intervals of 40 ft. along each wall. The lights on the opposite walls of each tube are staggered, thus furnishing a light every 20 ft. The light wires are carried through wrought-iron pipes, built in the walls, two for each line of lights, thus providing for two separate and distinct circuits. The system is arranged with alternate lights on the same circuit, so that in the event of the current on one circuit being off for any reason, the lights on the other will in all probability be in service. Connection with the wires is made through special steel outlet boxes, one for each light, built in the wall, through which the 1-in. pipes pass. The lights are about 7 ft. above the top of the bench-wall.

Conduits.—Conduits for telegraph, telephone, power, and lighting cables are built in the walls, as shown by Fig. 6. These duct lines have outlets through special cable shafts in the main shafts and through the walls of the open cuts to duct lines on the surface. The lighting cables through the ducts are on a 600-volt circuit, transformed to 110 volts at special chambers at stated intervals through the tunnel, where connection is made with the light wires.

Splicing chambers, at intervals of 400 ft., are built in the walls along the power, telephone and telegraph, and light-wire duct lines.

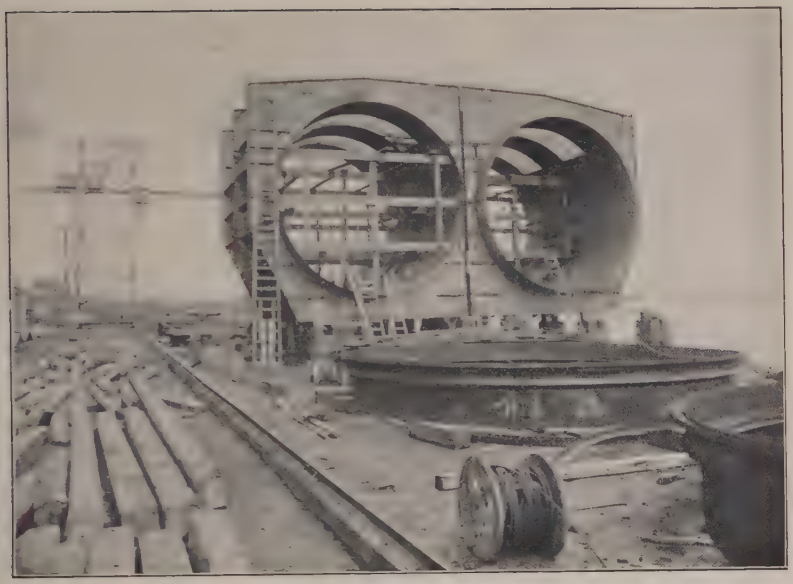


FIG. 1.—TUBES DURING ERECTION AT SHIPYARD.

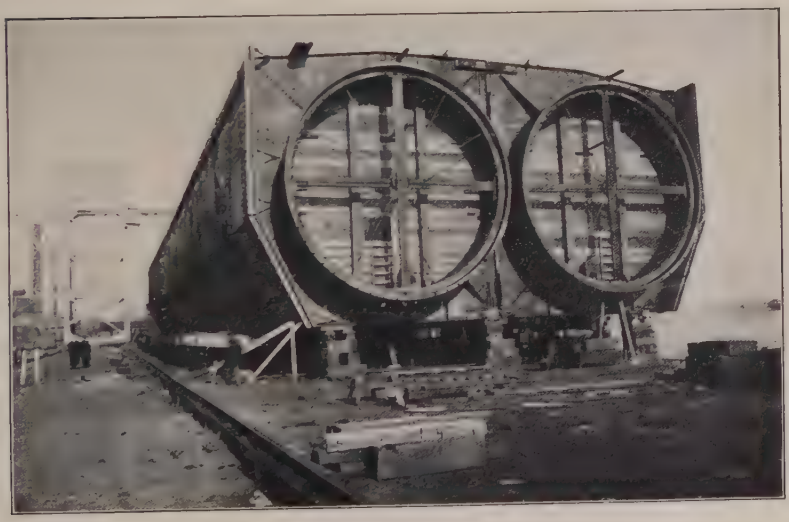


FIG. 2.—TUBES DURING ERECTION AT SHIPYARD

Drainage.—All water coming to the tunnel will be taken care of by five sumps, one in each portal, one in each approach near the river, and one in the subaqueous section, under the middle of the river, their capacities ranging from 30 000 to 40 000 gal. each. For the discharge of the water, each sump is provided with rotary pumps driven by electric motors. Under the center of the track, leading to the sumps through each tunnel, is a ditch in the concrete, 6 in. deep in the approach tunnels and 10 in. in the subaqueous tunnels, as shown in the tunnel cross-sections. It is calculated that these ditches are of ample capacity to take the maximum flow of water during the heaviest rainfall which is likely to occur. Although the capacity of the sump in the subaqueous tunnel is 30 000 gal., it is not expected that very much water will ever reach it, as there is practically no leakage in the tunnel, and most of the water, even during the heaviest rainfall, will probably be intercepted by the sumps in the approach tunnels. The details of the subaqueous sump are shown on Plate L.

Ventilation.—As all trains through the tunnel will be operated by electric locomotives, no special system of ventilation has been provided.

Track Construction.—The track construction in the tunnel differs from the ordinary type in that the ties are not continuous from rail to rail. Each rail rests on ties 3 ft. long; their outer ends are 19½ in. from the gauge of the rail and the inner ends at the edge of the gutter. Every fifth tie on the right-hand side, in each tube, looking in the direction of traffic, is extra long, to provide support for the third-rail. The ties rest directly on the concrete invert, which was built up to the grade of the bottoms of the ties. Each tie is laid on a bed of mortar $\frac{1}{16}$ in. thick for bonding purposes, and the ties are held in place by concrete placed between them to a depth of 5 in. at their outer ends, sloping toward the gutter to a thickness of 4 in. Bond between this concrete and the invert proper is furnished by dowels.

Before the tunnel was built, a short section of the Michigan Central Company's main-line track was changed to conform to the design proposed for the tunnel. This test section of track proved highly satisfactory, and the design was accordingly approved and adopted.

The standard tie used in the tunnel is 8 by 11 in., and the rail is the 100-lb., Am. Soc. C. E. section.

The general type of track construction, where separate ties are used for each rail, is not at all new, having been used years ago on some of the first railroads built in the United States. The details of the design for this permanent track construction were developed in the offices of the Tunnel Company. Views of the track are shown on Plate LI.

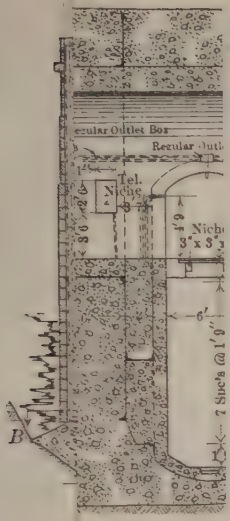
Fire-Hydrants.—The tunnel is provided throughout with a 6-in. water line, with hydrants every 100 ft., for protection against fire. Alarm boxes are located at intervals of 100 ft., with connection to the sub-station where the alarm is sounded.

Electrification.

After a thorough analysis of the operating requirements and the various systems of electrification, the continuous, or direct, system was selected as best suited to the conditions.

On account of the rather poor load factor, due to high peaks caused by the hauling of 1 800-ton trains on ascending grades, and then periods when no traction load is on the system, a careful study was made in order to decide on the most economical and reliable supply of energy. It was found better to enter into a contract with the Detroit Edison Company for the purchase of primary current than to erect a generating plant. The rate accepted required a practically constant load on the Edison Company's plant, and necessitated the erection by the Tunnel Company of a sub-station costing approximately $260 000. Its location is near the Detroit shaft, on the river front, and about 3 miles from the power-house, to which it is connected by three cables, any one of which has ample capacity to furnish the three-phase, 60-cycle, 4 400-volt current sufficient for the average 24-hour load on the sub-station.

The sub-station equipment for the beginning of operations consists of two 1 000-kw. motor-generator sets, a 2 520-ampere-hour, 650-volt, storage battery, with booster and regulating appurtenances, and the auxiliary equipment necessary to furnish power for lighting, pumping, signaling, etc. The storage battery, with its regulating equipment, keeps the input to the sub-station nearly constant, and, in conjunction with one of the motor-generator sets, will supply power for peak loads as high as 9 000 amperes.



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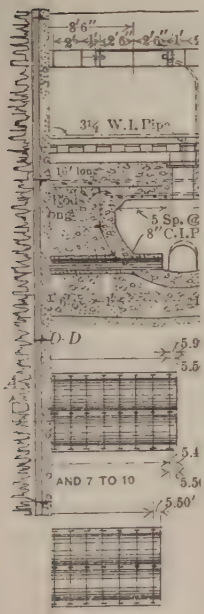

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D-D
5.9
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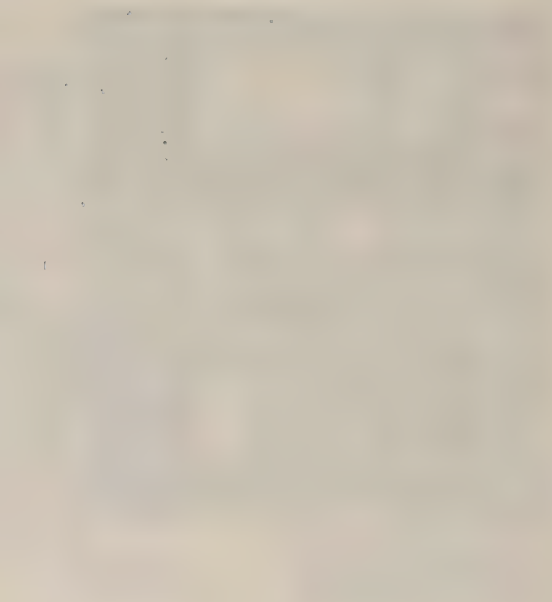
AND 7 TO 10
5.50'

NO. 6



The third-rail is of the Wilgus-Sprague under-running type, 70 lb. per yd., and its total length, for the initial installation, is 18½ miles. The distance from the east to the west end of the third-rail district is from Station 79 to Station 312, about 4.4 miles. At the shafts and summits, 650-volt feeders from the sub-station are joined to the third-rail. At the summits, where interlocking towers are located, sectionalizing switches have been installed, so that portions of the third-rail in either of the yards or the tunnels may be "cut out" for repairs, or in case of trouble.

In the tunnels provision has been made to erect an overhead contact conductor should it be desirable.

The locomotives were built by the General Electric Company, and exhaustive tests, on the first of the six ordered, demonstrated that the type is well suited for the operating requirements. The following is a brief description of the type of locomotive in service: General Electric Company's classification: 0440-E-200-4 G.E. 209A—which is the conventional method of stating that it is an articulated, four-axle type, weighing 200 000 lb., equipped with four 300-h.p. motors of the geared railway type.

Weight of one motor	10 560 lb.
Weight of all electrical parts	53 000 "
Weight of all mechanical parts	147 000 "
Weight on drivers	200 000 "
Diameter of drivers	48 in.
Rigid wheel base	9 ft. 6 "
Total wheel base	27 " 6 "
Gear reduction	4.37
Designed for trailing load: 900 tons on 2% grade at 10 miles per hour, with 600 volts at the third-rail.	
Maximum tractive effort	67 000 lb.
Balance speed on level, running light	35 miles per hour.

Each of the five sumps is equipped with duplicate sets of alternating-current, motor-driven, centrifugal pumps, and a third direct-current, motor-driven pump for emergency is installed at the Windsor portal sump to operate from third-rail current. The portal sumps have capacities considerably in excess of the requirements for taking care of any recorded rainfall, and the others are large enough to take

care of the water should the portal sumps, for any reason, be out of service.

The signaling is arranged so as to make one block from portal to portal, in each tunnel, and the four interlocking plants—two in each yard—operate the signals and switches electrically.

The yards are lighted by series arc lamps and the tunnels by incandescent lamps.

A fire-alarm and a telephone system have been installed, which provide means of quick communication between all the maintenance and operating men, to take care of regular work and emergencies.

Terminals.

When the location of the tunnel was approved, it was known that the approaches would destroy the efficiency, as then operated, of two Michigan Central yards, a small one in Detroit, and the terminal yard in Windsor. Steps were immediately taken to acquire property for a new yard at Windsor, and arrangements were made for the complete yard coincident with the completion of the tunnel. An ideal location, close to the tunnel summit, and just east of the old yard, was available, and as the land was practically all farm property, no extraordinary expenditure was necessary for real estate. In Detroit a somewhat different situation was presented. The main switching yard is about 3 miles west of the westerly summit of the tunnel, on the outskirts of the city. The right of way through the city is comparatively narrow, and, on account of the large number of adjacent industries it seemed to be out of the question, during the preliminary study, to widen or in any way enlarge the facilities of the railroad for any great distance without unwarranted expense, so that all plans were necessarily made and based on the right of way as then owned by the Michigan Central.

It was at first thought that the interchange of power in Detroit for tunnel trains might be made at the large yard west of the city. When the original report was made by the Engineering Committee of 1904, the feasibility of electrifying a small receiving and delivering yard near the tunnel summit, where electric and steam engines could be interchanged, was suggested, but the question being one of operation, was not determined at the time. It was apparent that some of the tracks through the city would have to be electrified, which immediately gave rise to the problem of the separation of street grades.

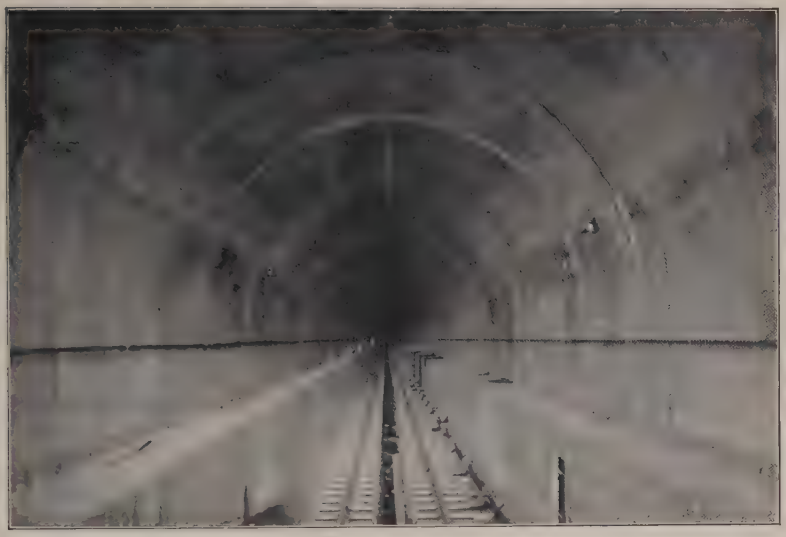


FIG. 1.—INTERIOR OF APPROACH TUNNEL.

FIG. 2.—TYPE OF TEST TRACK USED IN DETROIT RIVER TUNNEL.

West of the tunnel summit, the railroad crossed, at grade, a number of streets on which the traffic was very heavy, and, as the tunnel plans contemplated the use of an electrified third-rail, the Committee, for reasons which are obvious, urged strongly that the grades at all cross streets between the tunnel summit and the Junction Yard, West Detroit, be separated. Under an agreement with the City of Detroit, grade separation work of considerable magnitude had been in progress for several years in different parts of the city, and the recommendation of the Committee was promptly adopted.

On account of local conditions, subways are being built at all streets which are to remain open for traffic, some thirteen in all. Within the limits of this subway construction, some 2¼ miles, there are fifty or more industries served by thirty industrial tracks. The tracks have been elevated an average of about 6 ft., and all the work is being done without the purchase of additional property.

By the acquisition of some additional property immediately west of the summit of the tunnel, it was possible to arrange for an interchange yard to serve the requirements of the tunnel traffic, thus limiting the electrification of the tracks on the west to 24th Street. With the new layout of tracks, shown on the upper part of Plate LII, six tracks are reserved for freight movement through the tunnel, forming a combination known as a receiving and delivering yard. Each track has a capacity of 55 cars, 42 ft. long. Switches and cross-overs are provided, so that engines can pass quickly from the west-bound to the east-bound yard, and also from the freight yard to the passenger interchange yard. Near this yard, and convenient for the prompt handling of electric power, there is a small engine-house where electric locomotives are repaired. A new passenger station, between 15th and 17th Streets, is now under way, and until this is completed, tracks have been reserved for the exchange of electric and steam power at a point near 17th Street, from which point to the Third Street station trains are handled entirely by steam locomotives.

The New Station.—The present passenger station of the Michigan Central Railroad in Detroit is a little more than a mile east of the westerly summit of the grade of the tunnel, and its continued use would require the extra handling of all passenger trains between 17th and 3d Streets. A new station was therefore necessary, and careful study of the situation promptly determined the proper location and

care of the water should the portal sumps, for any reason, be out of service.

The signaling is arranged so as to make one block from portal to portal, in each tunnel, and the four interlocking plants—two in each yard—operate the signals and switches electrically.

The yards are lighted by series arc lamps and the tunnels by incandescent lamps.

A fire-alarm and a telephone system have been installed, which provide means of quick communication between all the maintenance and operating men, to take care of regular work and emergencies.

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FIG. 1.—INTERIOR OF APPROACH TUNNEL.

FIG. 2.—TYPE OF TEST TRACK USED IN DETROIT RIVER TUNNEL.

type. It was obvious that the proper location for such a structure, if conditions could be adjusted, was as near the summit of the grade of the tunnel as possible. The character of the passenger traffic on the Michigan Central governed in the selection of a through rather than a terminal station. The development of the plan for a through station in the new location involved the purchase of a large amount of property, but this has all been acquired, and preliminary work on the baggage, mail, and express-rooms and the station proper, is now well under way. The track arrangement for the new station is shown on the upper part of Plate LII.

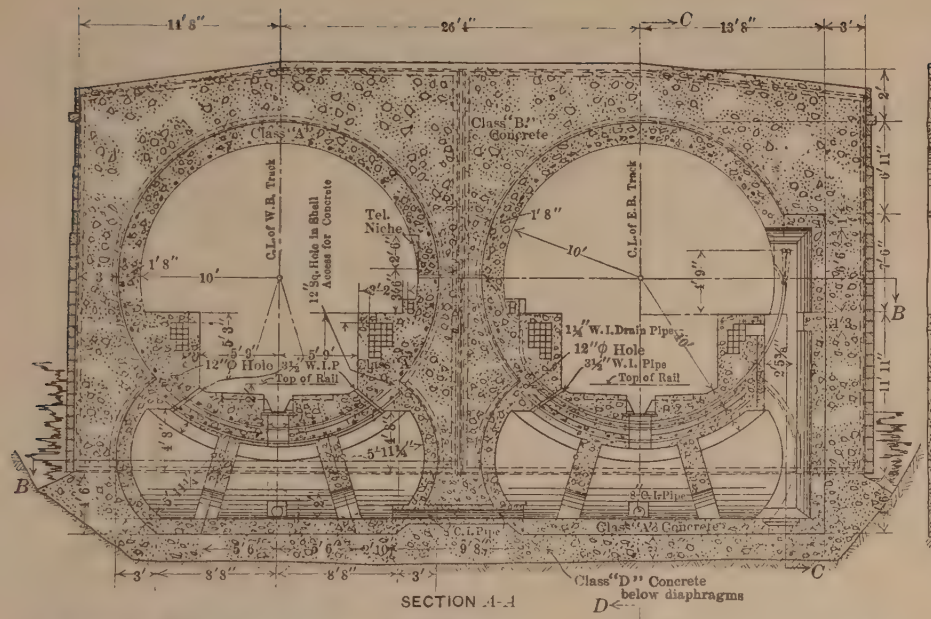
The presence of the third-rail in the station tracks was one of the governing elements demanding consideration, and, in connection with a comparison of street and track grades, forced a conclusion favorable to a subway handling of passengers to and from trains.

The Bush type of trainshed will be used, like that adopted for the Lackawanna Station, in Hoboken, N. J., and the new Chicago and North Western Station, in Chicago.

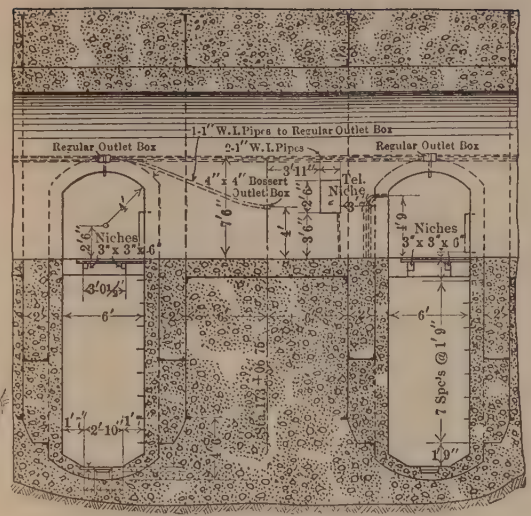
The Windsor Yard.—As previously stated, when the plans for the tunnel were being prepared, it was known that the easterly approach to the tunnel would destroy the Windsor yard, and a new location was accordingly selected, where a modern yard could be developed at a reasonable cost. The new location called for the purchase of a strip of land 500 ft. wide and 2½ miles long, immediately adjoining the railroad right of way on the north. This property was crossed by three highways; one was closed, one was diverted, and the third was taken care of by a subway. Several miles of new roads were built in lieu of the closed highway, thus caring for abutting property owners. The property was acquired after considerable opposition on the part of the owners and several condemnation proceedings, and the yard is now completed and in operation.

The arrangement of the yard is shown on the lower part of Plate LII. It is divided into three distinct sections: The electric interchange yard; the west-bound classification and east-bound receiving yards; and the east-bound classification and west-bound receiving yards. As a yard, it is modern in every respect, with a west-bound capacity of about 2 200 cars and an east-bound capacity of 3 000 cars. This does not include the electric receiving and delivering tracks, which pro-

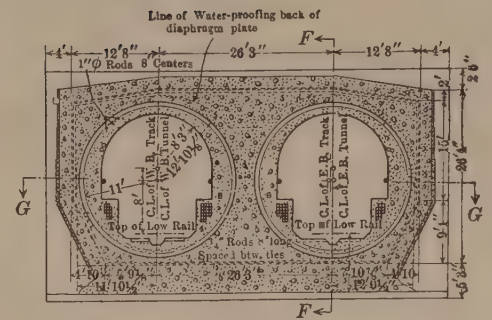
PLATE L.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXIV, No. 1212.
KINNEAR ON
THE DETROIT RIVER TUNNEL.



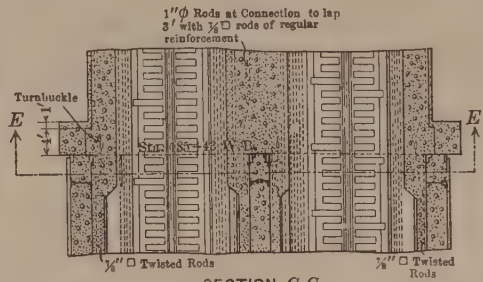
SECTION *A-A*


Regular Outlet Box
1-1" W.I. Pipes to Regular Outlet Box
2-1" W.I. Pipes
Regular Outlet Box
4" x 4" Bossert Outlet Box
Tel. Niche
Niches 3" x 3" x 6"
Niches 3" x 3" x 6"
7 Spcs @ 1'9"

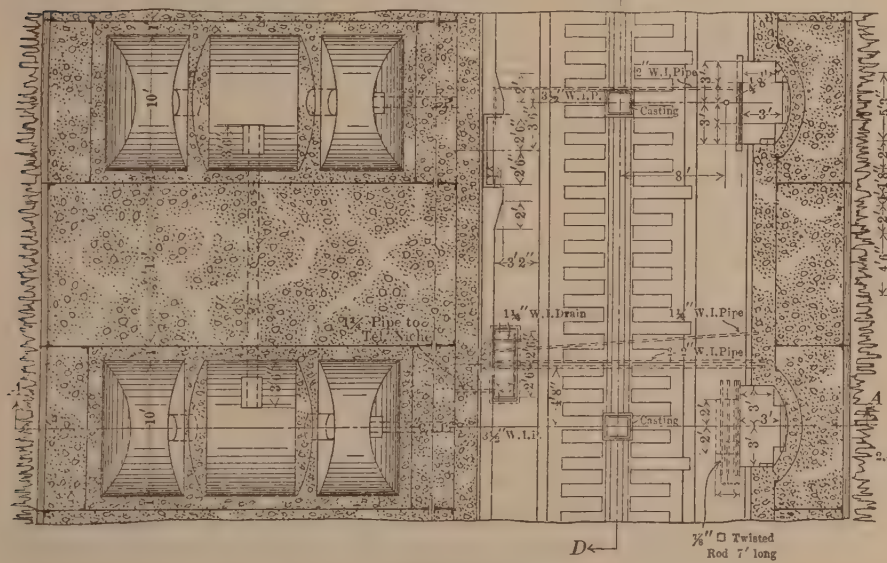
SECTION *C-C*


Line of Water-proofing back of diaphragm plate
1" ϕ Rods 8' Centers
C.L. of E.B. Track
C.L. of W.B. Tunnel
C.L. of E.B. Tunnel
Top of Low Rail
Rods 8' long
Space 1 btw. ties

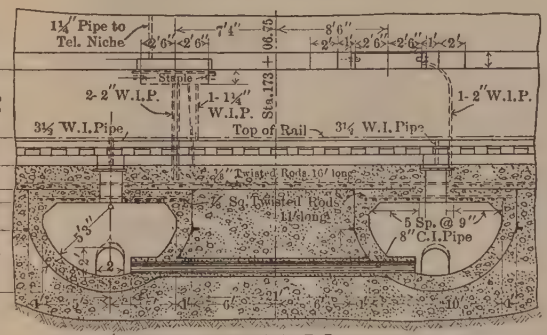
SECTION *E-E*



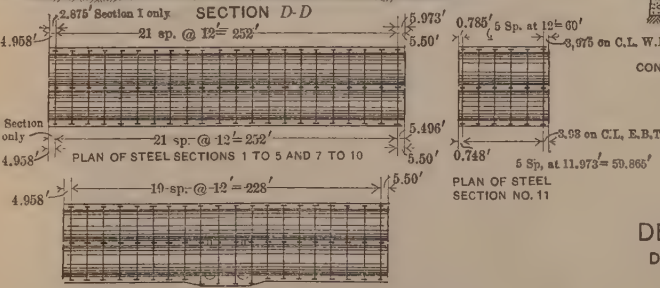
SECTION *G-G*



SECTION *B-B*
SUBAQUEOUS SUMP



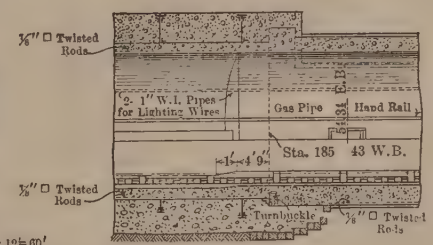
SECTION *D-D*



PLAN OF STEEL SECTIONS 1 TO 5 AND 7 TO 10

PLAN OF STEEL SECTION NO. 11

PLAN OF SECTION NO. 6



SECTION *F-F*
CONNECTION WITH EASTERLY APPROACH TUNNELS

DETROIT RIVER TUNNEL
DETAILS, SUBAQUEOUS TUNNEL

vide for the handling of the business of foreign roads as well as that of the Michigan Central traffic.

A new passenger station is being built in Windsor, east of the summit of grade of the tunnel, at the location shown on Fig. 2.

In Detroit the present arrangement of the tracks of the various railroads is such that any railroad can enter the new station over existing tracks. In Windsor it will be necessary to build a line connecting the several railroads. Such a line has been built connecting the Grand Trunk and the Michigan Central, which can be made to serve the Canadian Pacific and Pere Marquette also, thus providing an entrance to the tunnel for all the railroads.

Conclusion.

It has been mentioned that the contract for the construction of all sections of the tunnel, exclusive of the electrification and certain work of minor importance performed by the Tunnel Company, was awarded to the Butler Brothers-Hoff Company. The corporate name of this company was changed during the progress of the work to Butler Brothers Construction Company, Olaf Hoff, M. Am. Soc. C. E., retiring from the firm.

The construction contract was completed on July 1st, 1910, and the electrification work was so well in hand that the first train drawn by an electric locomotive passed through the tunnel on July 26th, 1910. Some little delay was suffered in the formal opening of the tunnel, due to terminal work in Detroit, but freight-train operations were started on September 15th, 1910, and the tunnel was opened for all classes of traffic a month later.

It is of more than passing interest and importance to record that the controlling lever of the electric locomotive used for the initial train movement through the tunnel was in the hands of Mr. Henry B. Ledyard. To Mr. Ledyard, more than to any one man, is due the credit for the existence of this structure, forming, as it does, a most important all-rail connection between the United States and Canada, and, what is of vastly greater importance to the commercial world, an all-rail connection between two parts of a great railroad system. Due to Mr. Ledyard's persistent and untiring efforts during the early stages of the river-crossing problem, and later his unqualified endorsement

of a recommended method of tunnel construction, novel and untried, pages may be added to engineering history.

It is with a sense of great obligation that the writer takes advantage of this opportunity to express his appreciation of the wise counsel and judgment of Messrs. Wilgus and Carson.

The contract was unique, particularly with reference to the subaqueous section, leaving the working out of details to the ingenuity of the contractor. The ability, energy, and resourcefulness of the contractor, as concentrated in the representative in immediate charge of the work, Mr. William Butler, was a source of constant gratification.

An able and loyal staff of assistants was found in Messrs. Benjamin Douglas, M. Am. Soc. C. E., J. C. Mock, A. C. Everham, Assoc. M. Am. Soc. C. E., William Burton, Assoc. M. Am. Soc. C. E., J. M. Hammond, and E. M. Walker, Assoc. M. Am. Soc. C. E., all of whom remained until the completion of the work.

DISCUSSION

H. A. Carson, M. Am. Soc. C. E. (by letter).—In 1845, De la Haye, of England, suggested making a submarine railway by constructing wrought-iron tubes above water in sections 400 ft. long, bulkheading them so that they would float, towing them to the tunnel location, then admitting water, and sinking them to a suitable bed. Mr. Carson.

Since then many inventors have patented or published more or less practicable variations of this scheme. For example, in 1869, Martin and le Gay, of France, proposed a tunnel between France and England, consisting of metal tubes sunk to the bed of the Channel and surrounded by concrete. They proposed a feature afterward used at Detroit, of definitely limiting the exterior concrete by using sheathed frames fastened to, and sunk with, the metal tubes. In the following year a similar scheme was proposed by Mr. Thomas Page, an Englishman, who had been connected years before with the Brunel Tunnel under the Thames.

The writer does not know of any actual construction that followed De la Haye's suggestions until M. Belgrande, in 1866, built a pair of small pipe tunnels for sewers under the Seine, in Paris. Each had a diameter of 1 m., was 156 m. long, and was made of iron plate. Since then there have been numerous examples, many of them interesting and ingenious, of iron water pipes sunk to the beds of rivers and harbors.

The first masonry tunnel of the De la Haye class, and also the first of this kind large enough for men to walk through erect, of which the writer knows, was constructed by himself, aided by Messrs. W. Blanchard, W. B. Smith, and other able assistants, when in 1893-94, he built, without the intervention of contractors, the Metropolitan sewer in the outer portion of Boston Harbor. The details in 1893 were slightly different from those in 1894; it will suffice to describe the latter. These tube sections were made of a combination of brick and concrete, with exterior wooden staves, 4 in. thick. The tube sections had external diameters of a little more than 9 ft., and were 50 ft. long. Temporary water-tight bulkheads were inserted at each end, and external flanges for bolting the contiguous sections end to end, were provided. These bulkheaded sections were tested for air-tightness, and inferentially for water-tightness, by exhausting the air and measuring the degree of rarefaction with a vacuum gauge. The weight of the sections relative to their displacement was such that they would barely float in sea-water. They were made in cradles, above water, alongside of a wharf. After completion and bulkheading, they were lowered by long vertical screws, moved by steam power. The sections were then towed from $\frac{1}{2}$ to $\frac{3}{4}$ mile to their position for lowering, and,

Mr. Carson. by admitting water, were sunk to saddles accurately fixed in a trench dredged in the gravel of the harbor. When a section had been laid within a few inches of its predecessor, the two were bolted together, end to end, by divers. A rubber gasket was provided at each end between the flanges. The subsequent operations consisted of back-filling the trench around and over the pipes, removing the bulkheads successively, beginning at the land sections, and pumping out the water. At each joint, between two consecutive sections, a short closing length of masonry had to be put in to make good so much of the interior adjacent walls as had been occupied by the bulkheads. This was accomplished without difficulty as the rubber gaskets had made the joints perfectly water-tight. As these gaskets were outside the masonry, the fact that they would become disintegrated in time was not a matter of consequence. It was found that the whole tunnel, 1 500 ft. in length, was perfectly water-tight at the joints and elsewhere, true to line and level, and satisfactory in every way.

When the writer became a member of the Advisory Board of the Detroit Tunnel, he proposed a similar method, referred to by Mr. Kinnear, as Design *B*. He did not propose an outer surrounding volume of concrete, on account of its expense. He admitted, however, that a large expense was justifiable in protecting a tunnel the top of which was brought up to the bottom of a river having such a heavy floating traffic. The possibility of a foundered vessel resting on the tunnel was constantly kept in mind by the Advisory Board.

The scheme proposed by Mr. Wilgus at Detroit was not merely a reinvention of that of Martin and le Gay. It possesses in addition thick inside walls and other important features.

In 1877, a well-known engineer, William Sooy Smith, M. Am. Soc. C. E., proposed to construct a tunnel under the river at Detroit by sinking pneumatic caissons such as used in making bridge piers. These caissons were to touch one another, end to end, under the river, and in them the tunnel was to be built. It is an interesting fact that the general features of the French scheme of Martin and le Gay was recently used at Detroit at the same time that the basic features of the American plan of Sooy Smith was being used in Paris, in connection with the crossing of the Metropolitan Subway under the Seine. Each scheme had been re-invented, with important modifications.

Mr. Dunham. H. F. Dunham, M. Am. Soc. C. E. (by letter).—Comments relating to the details of the work described in this remarkable paper will merely emphasize the ingenuity, boldness, and successful completion of the project as a whole.

Had it been desired to check further the elaborate and rather expensive transfer of elevations across the river, half a dozen lengths of ½-in. pipe and two similar metal floats costing 50 cents, more or

less, might have been used, and the averages of a few simultaneous readings on opposite sides of the river duly recorded. These averages might have been taken with confidence in place of the high-grade instrument work, and, like that work, found to be corrected automatically for curvature, radiation, and instrumental errors. Mr. Dunham.

Readers of the paper will wish the author had given more space to a description of the steps by which the methods suggested originated or were shaped up for actual work; and to an explanation of why one method instead of another was finally adopted.

No one feels more keenly than the writer the great difference between the description of a scheme and of an accomplished work. The task performed is the scheme embodied and made permanent. That should satisfy. Yet there is a relation between methods in novel work which, for its real value, must depend in part on the work done. This might be illustrated by inquiring whether the author now considers Design *A* superior to Design *B*. It often happens that one part of a piece of work is more, and another part less, difficult than a builder anticipated. Has experience in connecting comparatively thin-walled sections made it evident that sections with thick walls would be unmanageable?

In sinking to its position, during a "June rise" in the Ohio River, a heavy timber grillage with a coffer-dam and many courses of pier masonry for a 525-ft., double-track railway span, the writer found that it was possible to hold and control a large unit in a swift current. From this experience came ideas of tunnel sections, and, for one gravelly location, plans were made and protection obtained to promote the interests of a company. The work, however, was not undertaken, and no reference would be made to it here except that the methods and claims intended to further one piece of work (and not to interfere with another, since that would be unprofessional), have much in common with the work under discussion.

The double or twin-tunnel section, with transverse working openings through which concrete or other material could be deposited to secure a good foundation, was illustrated and claimed. The use of dowels to draw the sections together, the bulkheads, and the methods of unwatering were the same. In one or two respects, however, there were differences: It was proposed to lower a section, and, after making it secure and firm, to use it in lowering and guiding another section into position; each section was to be heavy enough to withstand the exterior pressure; and the work was to be done with little aid from divers. The details for a change of grade without a change in the method of holding one section firmly to another were worked out; also the plan of laying the sections each way from mid-stream and into comparatively shallow water where possible.

Mr. Dunham. An interesting feature in the description of the work done by the author is the use of air cylinders instead of boats or scows to control a section as it was being placed. Were the cylinders used on account of cheapness, or to give better control?

The excellent quality of the concrete was due, in part, at least, to the pressure, and filling of voids in the tubes, and from the pressure of the water above when the concrete was finally deposited.

The reason for filling one or two sections with solid concrete is not fully explained; but it is far easier to ask questions about the work than it is to formulate in word the credit which the design and its execution merits.

Mr. Buerger. Charles B. Buerger, Assoc. M. Am. Soc. C. E. (by letter).—The unwatering of the river sections of the tunnel after the completion of the concreting, as described by Mr. Kinnear, would seem to permit of a simpler method.

Provide each section at its lower end with an outlet pipe, say, 6 in. in diameter, starting near the bottom plates, and leaving the tube through the top, with a valve above the concrete line. Make a hose connection to the top 2-in. air-escape valve used in sinking the tube, and force air into the tube by a compressor. Each section could be unwatered independently of the others with only a 2-in. hose as connection. A whole section would be emptied in 24 hours (not allowing for leakage) by a small compressor with a capacity of 250 ft. of free air per minute. With a pressure difference of only from 0 to 10 lb., between the inside and the outside of the tube, a generous allowance for leakage would not increase the requirements to more than 500 ft. of air per minute, and the whole of this discharge could pass through the 2-in. hose. The additional power required would not seem to be a serious objection.

Mr. Walker. E. M. Walker, Assoc. M. Am. Soc. C. E. (by letter).—The writer, having been in charge of the survey and alignment work on the Detroit River Tunnel, is especially interested in Mr. H. F. Dunham's statement that water gauges might have been used for transferring levels across the river, instead of instrumental work. In fact, such an idea had occurred to him, but he did not believe that float gauges of sufficient and equal sensitiveness for the purpose could be devised without considerably greater expense than the 50 cents estimated by Mr. Dunham, and also in view of the constant disturbance of the surface by the large volume of vessel traffic, and the further possibility of error due to the much greater current velocity on the Canadian side of the river, and the idea was abandoned. The instrument work was not as expensive as the description indicates, for, according to the writer's recollection, the actual time chargeable to this work was not much more than about a day for four men, which, at the salaries paid, would

make the cost ten or twelve dollars. Confidence in the accuracy of the result was, in the writer's judgment, well worth this expense. Fixed gauges, graduated to tenths, were used to control the dredging, and these gave an approximate check, but these readings, although rough, seemed to have enough variation to justify the instrumental method. Mr. Walker.

In reference to the measurement of the base lines for the final triangulation, part of the measurement of the Detroit base line was done with the tape supported only at the ends under a tension of 24 lb. The 100-ft. points were indicated by plumb lines of No. 30 hard brass wire. This method was used over ground where it was impracticable to maintain a line of supports, and was adopted as most expedient. To the surprise of the engineers, the measurements checked more closely than when the tape was supported, due apparently to eliminating the friction on the supports. This accounts for the lower variation in the measurements of the Detroit base line, as noted in the paper. Since making these measurements the writer has been pleased to note several instances where accurate measurement has been done successfully in this manner, so that the method now appears less radical than it seemed at that time.

Olaf Hoff, M. Am. Soc. C. E. (by letter).—When the firm with which the writer was connected in February, 1906, received invitations to submit proposals for the construction of the Detroit River Tunnel, he immediately and with assiduity set to work on this intensely interesting problem. At that time he had no knowledge of the numerous patented inventions for building subaqueous tunnels in a trench in the bottom of a river or waterway, and, in fact, was under the impression that the Board of Engineers was the first to suggest this unique scheme. Later, having occasion to look the matter up, he was surprised to find a number of patents on such tunnels, mostly impracticable schemes, of doubtful merit, not one of which was ever carried out. Mr. Hoff.

Little or no attention was given to Design *D*, which would be eliminated as less advantageous, provided it were possible to build the tunnel in a trench, as contemplated by Designs *A*, *B*, and *C*, submitted by the Board of Engineers.

Accordingly, a careful study of these was made to determine their practicability, and especially to discover the objectionable features, if any, which might make their execution difficult, hazardous, or unreasonably expensive.

These designs are shown in cross-section on Plate LIII. They contemplated a tunnel 2 ft. less in diameter than the one built. In the specifications sent out to bidders, the designs were described as follows:

Mr. Hoff.

"Design *A* may be briefly described as a method by which the pervious material underlying the river is replaced by an imperfectly water-tight material of concrete placed under water, through which, without the use of shields, the inner tubes or tunnel proper may be constructed so as to secure water-tightness, continuity of strength, and good workmanship, with an absence of high air pressures."

"Design *B* may be briefly described as a method by which the inner tubes of the tunnel proper are first built in floats or on dry land, floated and sunken into position on a proper foundation, and joined together by a suitable method so as to secure continuity of construction and water-tightness."

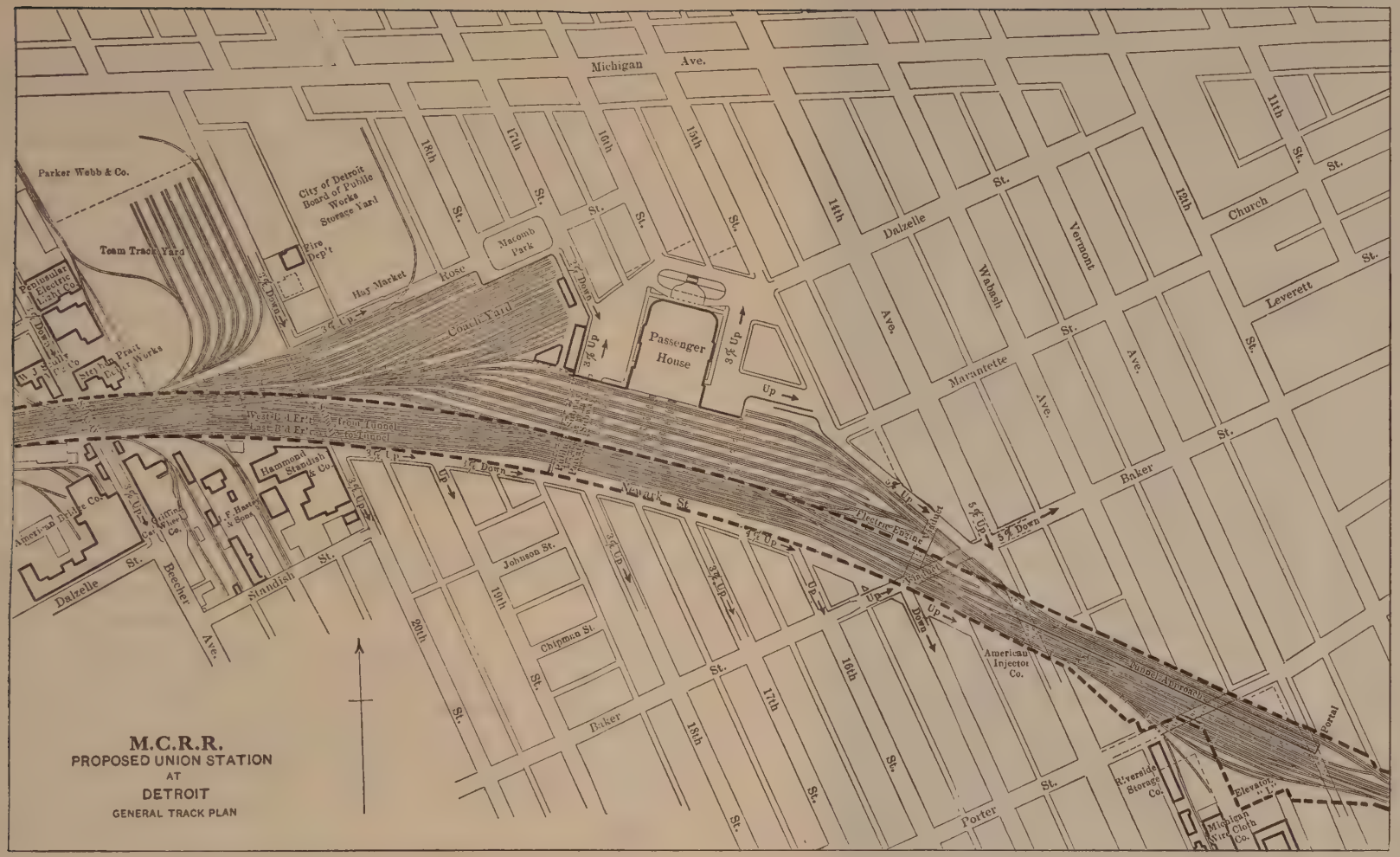
"Design *C* is practically the same as Design *A*, with the exception of the details shown on the plan, which prescribe a certain style of steel form and arrangement of reinforcement that differ from the arrangement shown on the plan for Design *A*."

The specifications for Design *B* further required the joints to be "made at the connection of the tubes so as to insure water-tightness and an equality of strength with the remaining portions of the tube, so that the joint will not be an element of weakness in the structure, and will be fully as strong as any portion thereof."

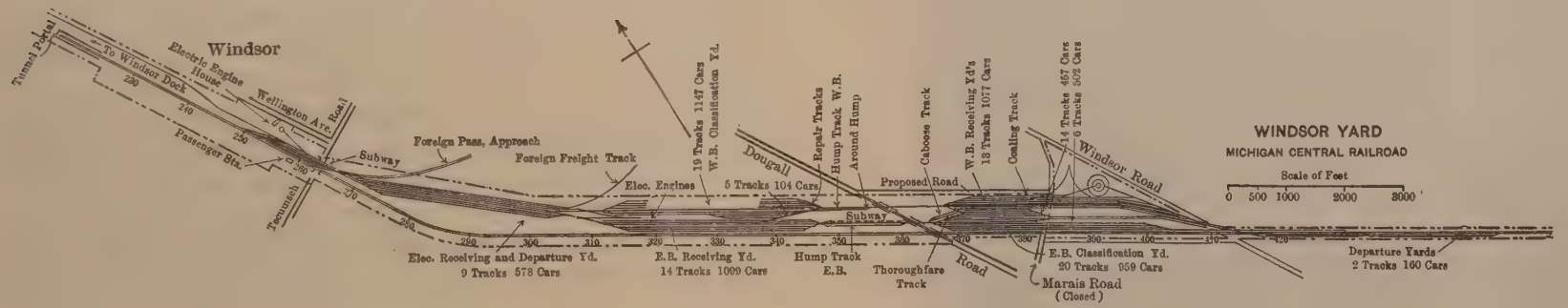
Some of the considerations and objections brought out by a study of these designs were as follows:

Design A.—This design, it will be noticed by reference to Plate LIII, contemplated filling the whole trench with concrete surrounding a series of cores or forms, and forming a kind of working chamber, inside of which the tunnel structure proper was to be built of reinforced concrete. The cores or forms might be of wood or steel, and were to remain in place permanently if of steel. A small air pressure was thought necessary, especially if the forms were to be of a temporary nature. The slopes of the trench, as shown, were $\frac{1}{2}:1$.

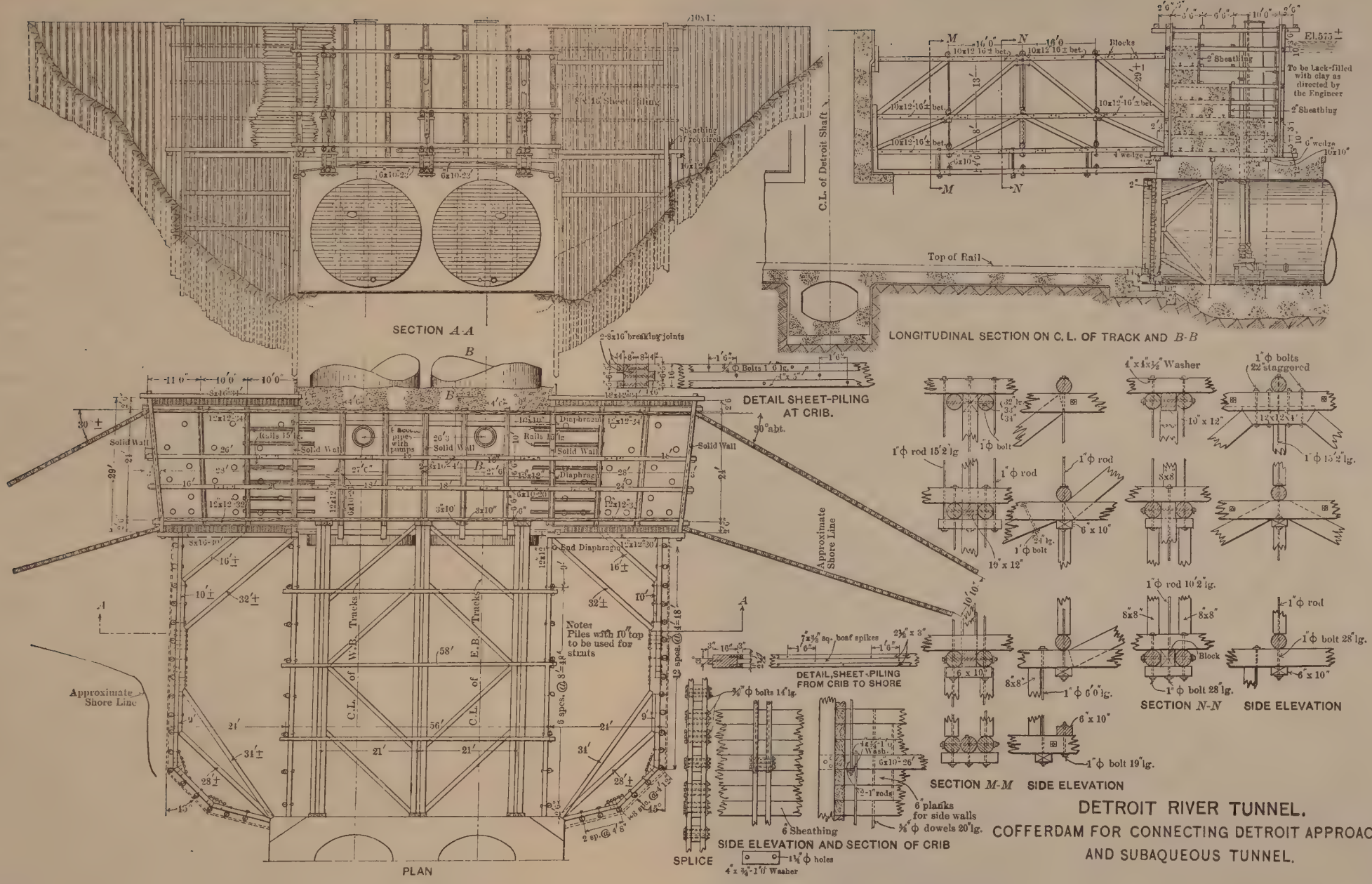
The writer came to the conclusion that this design was impracticable of execution. Depositing concrete as proposed in an open trench under water would have resulted in a very poor and unsatisfactory product, judging from such rather limited experience in that respect as the writer had had at that time. The experience subsequently gained has amply demonstrated this. It was evident, therefore, that the use of compressed air would be necessary, and that it would have to be at a pressure equal to the full hydrostatic head at the bottom of the tunnel, or about 30 lb. per sq. in. Later, this pressure was actually found to exist, as stated in the paper, even with the excellent grade of concrete obtained, which would have been absolutely impossible of attainment in Design *A*. Such a pressure would have produced a resultant internal tension in the concrete at the roof, which in all probability would have caused disastrous "blow-outs." About the only practicable way would have been to place the concrete in bags, which process would have been both slow and expensive, on



M.C.R.R.
PROPOSED UNION STATION
AT
DETROIT
GENERAL TRACK PLAN



WINDSOR YARD
MICHIGAN CENTRAL RAILROAD
Scale of Feet
0 500 1000 2000 3000



DETROIT RIVER TUNNEL.

COFFERDAM FOR CONNECTING DETROIT APPROACH AND SUBAQUEOUS TUNNEL.

account of the necessity of using divers in placing them. If deposited by tremie or bucket, the concrete would have flowed lengthwise of the trench, permitting the water to cut out more or less of the cement. Difficulties would have been encountered in forcing the concrete under the forms, as it would naturally move along the line of least resistance, which would be lengthwise of the trench. Mr. Hoff.

A great objection to Design *A* was the indefinite and excessive quantity of concrete required to fill the trench. No reliance could be placed on the clay standing at a slope of ½:1, the probability being that 1:1 would be about the best that could be secured. As demonstrated afterward, the actual slope varied from 1:1 to 1½:1, the average being nearer the latter than the former. In addition there was the question of how close to the actual line it would be possible to dredge—perhaps within 5 or perhaps even 10 ft.—at any rate, again greatly adding to the quantity of exterior concrete. A subsequent computation, based on the actual dredging done, showed that at least 80 000 cu. yd., and more probably 100 000 cu. yd. (or even more), would have been required in addition to the quantity actually used in the adopted design (about 102 000 cu. yd.). This would have represented from $500 000 to $600 000 in money, and at least another year in time.

Design C.—This was open to the same objections as Design *A*, with respect to the depositing and the quantity of exterior concrete. It had the advantage of safety, however, as it involved the definite use of a steel shell for an inner and permanent form, which was reinforced with 10-in. curved I-beams, 5 ft. from center to center and riveted to the shell along its circumference, thus making it of sufficient strength to resist the water pressure. The estimated cost of the steelwork in this design was quite high, and it would have been difficult to place the interior concrete on account of the I-beams.

Design B.—This design contemplated sinking in the trench sections of the completed tunnel of reinforced concrete, similar in construction to the interior structure built inside the exterior concrete of Design *A*, but 30 in. thick, instead of 23 in. When in position in the trench, the lower half was to be embedded in concrete and the upper half covered with back-filling. This design would probably have been more economical than either *A* or *C*, but it offered these difficulties: The tunnel would probably have to be built in 50-ft. or 100-ft. sections, in order that they might not break in two in handling. Their weight per foot would have been more than ten times the weight of the tubes of the adopted design while afloat, with a much less power of resistance in case of plunging during sinking. The concrete when submerged would have absorbed water, making the weight of the tubes variable and thus adding to the difficulty of sinking. There would have been

Mr. Hoff. a great number of extremely difficult and expensive joints, and it would have been especially difficult to make them of equal strength with the tubes. Considerable time would have been required to build the sections; if built in 100-ft. lengths, 52 would have been required, and each would have needed at least 2 months for building and hardening. Even by constructing a plant to build four at a time, 26 months would have been required, including both winter and summer. There would have been some difficulty in placing the concrete under the tubes in the trench, in order to give them a good bed. Probably this could have been done best by the use of bags and divers.

The result of the foregoing analysis was the gradual development of the design submitted by the writer, which, together with the specifications and the accompanying proposal, was accepted by the Board of Engineers, and according to which the tunnel was built, the contractors assuming full responsibility for its success or failure.

The first object sought in working out this design was the elimination of compressed air, with its attendant cost and restrictions in prosecuting the work. Nevertheless, as a matter of prudence, it was thought well to be prepared to use it, in case the exterior concrete, contrary to expectations, should prove a failure. The initial step toward the accomplishment of this was a tube of steel of sufficient strength in itself, or in connection with the exterior concrete, to resist the water pressure and effectively to prevent its ingress into the tunnel, thus saving the cost of water-proofing. This shell, at the same time, would constitute an inner form for the exterior concrete; if lined with concrete, the shell could be made quite thin; if not lined, it would naturally be made thicker.

Prudence required that, in this case at least, being the first time the scheme was tried, a substantial inner lining be provided, in case the quality of the exterior concrete should prove disappointing. Assuming this concrete to be of good quality (which was so thoroughly established later during construction), the steel shell need only be of sufficient strength to resist the hydrostatic pressure, assuming the concrete to hold the shell firmly in place and prevent buckling. It was proportioned on this assumption.

The next object sought was to reduce the exterior concrete to a definite quantity—the minimum required—without filling up the whole trench, thus saving a large item of cost. A little study and a few calculations soon demonstrated that this minimum would be the quantity necessary to overcome the buoyancy of the mass in the trench, when the tubes were unwatered, and prevent them from floating up again. This condition would obtain with the exterior concrete formed in the shape of a rectangle of such dimensions as to give a minimum thickness of 3 ft. around the steel tubes at the top, bottom, and sides, the distance between the tubes also being 3 ft. In order to secure

this result, it would be necessary to have an exterior form for confining the concrete, preferably of wood, to make it as inexpensive as possible. Portable forms of wood or steel were considered, but the idea was soon abandoned, as very little, if anything, would be saved in the cost, and the operations would have been more complicated. Mr. Hoff.

The next step was to secure this outer form to the steel shell or inner form. Wooden braces were first thought of (to save expense), but were abandoned because they would be buried permanently in the concrete. A solid steel plate seemed to be the proper solution, as this would divide the tubes longitudinally into compartments, which could be filled with concrete, one at a time. This plate need be of the smallest thickness practicable, and, of course, would require to have its edges stiffened with angle irons, the wooden form or sheeting being bolted to those on the sides. Thus the diaphragms were developed, together with the pocket or compartment principle, to which, in the writer's judgment, the success attained at Detroit is to be attributed. The concrete was discharged through the tremies under a head; its lateral flow was confined to the exterior sides of the compartment, and thus it was forced under the steel tubes, affording them a reliable and satisfactory bearing. The steel diaphragms were 12 ft. apart, thus greatly stiffening the steel tubes; and the angle iron flanges along the edges obtained a good grip on the concrete, binding the whole together in one composite structure of more than sufficient strength to resist all external or internal forces to which it might be exposed.

This design, as adopted and carried out, has frequently been referred to as a "Modification" of Design *A*. By a comparison of the designs, any one interested can form his own judgment as to what extent it is a modification of Design *A* or an original and independent design.

Details of Design.—As previously stated, the steel shell was proportioned to resist the full hydrostatic pressure, thus assuming that the water would penetrate the exterior concrete, but that the latter would be of sufficient strength to prevent deformation of the shell. This was made of $\frac{3}{8}$-in. plate, with a maximum stress of 9 400 lb. per sq. in. gross, or 13 200 lb. net (deducting for rivet holes). The rivets were $\frac{3}{4}$ in. in diameter, 3 in. apart, and arranged in two rows, also 3 in. apart. Each rivet sustained a maximum shear of 6 200 lb.

The length of the steel tubes was fixed at 262.5 ft., being one-tenth of the original length of the subaqueous tunnel (2 625 ft.), so that, when down in the trench and resting only on the platform supports at the ends, in case of an accident which might cause the air cylinders to be filled with water, the tubes would span across from end to end without collapsing, subjected to an extreme fiber stress of 16 000 lb. per sq. in. This contingency, however, never arose.

The thickness of the wooden sheeting was determined on the

Mr. Hoff. assumption that the concrete could not possibly exert a greater pressure on it than its weight in water (taken at 75 lb. per cu. ft.) multiplied by its height in the pocket, thus acting as a liquid, and that, if this pressure were kept just below the breaking strength of the timber, the sheeting would be safe. The extreme fiber stress varied from 5 735 lb. per sq. in. on the 6-in. planks at the bottom to 2 670 lb. on the 4-in. planks 6 ft. from the top. No planks were ever broken, notwithstanding the fact that the cheapest lumber obtainable was used for this purpose.

The bulkheads at the ends of the tubes were placed at the first diaphragm from the end, so as to obtain a rigid frame against which to caulk along their circumference. They were made of yellow pine, which would swell less than any other wood when soaked. The timber in the full-pressure bulkheads was subject to an extreme fiber stress of 1 600 lb. per sq. in. The total pressure on each of the latter, when the tubes were unwatered, was 775 tons, which was distributed as evenly as possible over the steel shell by using suitable braces reaching back to the next diaphragms.

The semi-bulkheads for the first section were made 7 ft. deep at the center, but this was not quite sufficient, and, for the remaining sections, they were made 9 ft. deep, which gave satisfactory results. These semi-bulkheads were introduced as a result of some experimental tests, made by the writer with a tin model of a tube section, in order to study the possible tendency to plunge during the process of sinking. It was found that this tendency would not manifest itself until the tubes were about two-thirds or three-fourths filled with water; the object of the semi-bulkheads was to trap the air at this stage, in case one end should fill ahead of the other, and thus check it until the other end caught up.

One peculiar feature of this method of tunneling is that the load on the bottom of the trench during construction will be as great as, or greater than, the maximum load of the completed tunnel when in use. In other words, the weight of the water inside the tubes is equal to, or greater than, the weight of the concrete lining and the live load. At Detroit the load on the bottom before unwatering was 1 410 lb. per sq. ft., and the load of the completed tunnel plus a live load of 6 000 lb. per lin. ft. of each track was 1 400 lb. per sq. ft.

As stated previously, the quantity of exterior concrete needed was just enough to overcome the buoyancy of the whole mass when the tubes were pumped out. Owing to uncertainty as to the quality of this concrete, especially at the top of the tubes, where the water flowing over the concrete might be expected to wash out some of the cement before it had set, the thickness above the tubes was increased by 1½ ft., making the minimum 4 ft. 6 in. The same quantity of concrete was

deducted at the sides of the tubes by sloping the lower 11 ft. 4 in. at the rate of ½:1, as shown on the plans. Mr. Hoff.

Adding the concrete on top was an unnecessary precaution, as its quality, to the very top surface, proved to be so good. It would have been better not to have sloped the sides, but to have retained the straight rectangular shape of the exterior concrete, and to have made the thickness on top about 6 in. greater than at the sides, on account of the top surface being likely to run uneven to that extent.

When the tubes were unwatered, the relation of weight to buoyancy was as follows:

Weight of steel shell, per lin. ft.....	4 200 lb.
Weight of exterior concrete, at 140 lb. per cu. ft. (excluding that below the diaphragms).....	107 600 "
Total weight.....	111 800 lb.
Displacement	101 500 "
Excess weight per linear foot preventing the tubes from floating up.....	10 300 lb.

Of the greatest importance was the problem of lowering the tubes into the trench and keeping them always under absolute control. To this end the four air cylinders were devised, and served the purpose most successfully. They were of such size as to have a combined buoyancy, when submerged, slightly in excess of that required to hold a tunnel section in suspension. The total buoyancy of the four air cylinders was 609 tons, and the total weight of a submerged tunnel section was 516 tons, leaving an excess of 93 tons, or 23¼ tons for each cylinder; accordingly, the corresponding quantity of water, would have to be admitted to the center or adjustment compartment of each, in order to admit of lowering the mass into the trench. They were placed in wooden cradles on top of the diaphragms and attached by flat straps to the third and the sixth diaphragms from the ends; these straps passed over the cylinders and had turnbuckles for tightening, and links which engaged pins through the diaphragms, which were suitably reinforced for the purpose, so that divers could readily disconnect the cylinders when needed for sinking the next section. A set of timber braces was placed longitudinally between the cylinders, and the tubes were designed to hold them firmly in place, in case the tubes should take a plunge while being filled. The straps were placed around the cylinders at such points as to produce a minimum longitudinal bending, to resist which a riveted lattice truss was placed vertically in the center line of each cylinder and made a part of the shell.

These air cylinders also served another purpose, namely, as compressed air receivers. Immediately before sinking they were pumped full of compressed air (under a pressure of from 30 to 40 lb.) and

Mr. Hoff. connected with the 2-in. air valves at the top of the end bulkheads by a rubber hose having a cut-off valve. If the air trapped between the semi-bulkheads and the end bulkheads escaped, on account of leaks, it could be replaced from the air cylinders by simply turning the valve and admitting as much as would be necessary to check or even raise the end which was going down too fast.

When the tubes had been landed in the trench and concrete had been placed under them at the ends and at least at one point in the middle, so as to give them not less than three points of support, the air cylinders could be released. This was done by first filling them with water, which caused the weight of the tubes to be gradually transferred from the cylinders above to the concrete below. When filled, the stresses in the attachments were neutralized and the cylinders were easily disconnected by divers and brought to the surface, either by forcing air into the center compartment or raising them by derricks. Both methods were used.

A word may be said on the subject of holding the tubes against the current during sinking. The tubes and air cylinders when submerged occupied one-eleventh of the area of the river cross-section; accordingly they dammed the river to that extent and increased correspondingly the velocity of the current. This was ordinarily 3.4 ft. per sec. at its swiftest, and was probably increased to 3.7 ft. while the tubes were submerged and until they were landed in the trench. It was estimated, using the following formula, that the pressure against a tube section and its air cylinders would amount to approximately 80 tons, and the anchorage cables and tackle were proportioned accordingly. That this assumption was very nearly correct was unexpectedly proven one day, when the superintendent tried a 1¾-in. line in the blocks instead of a 2-in. line, because the latter worked too hard in the sheaves. The 1¾-in. line snapped, while the larger one held; the breaking strength of the former was about 70 tons and of the latter about 90 tons, both being eight-part lines (running over four sheave blocks). The resistance, r, in pounds per linear foot $= \xi \frac{w}{g} d \frac{v^2}{2}$, in which $w =$ the weight of the fluid, in pounds per cubic foot; $d =$ the diameter, in feet; and $v =$ the velocity of the current, in feet per second. $\xi = 1.3$ for flat surfaces, and 0.7 for cylindrical ones.

The plans originally contemplated the use of anchors of cyclopean concrete planted in the bed of the river, to which the anchor lines were to be fastened. The superintendent entertained the idea, however, that ordinary ship anchors would suit the purpose equally well; they were tried, but failed utterly. For the third tube sections as many as twelve such anchors were laid out and connected up with lines, but they slipped, and the tube section swung down stream. Thereafter the method described in the paper was used (planting a

concrete slab in a hole dredged out in the river bottom and filling clay on top), and proved very satisfactory. Mr. Hoff.

The time required for taking a section from its moorings, placing it in position over the trench, attaching it to the anchor lines, filling the tubes, adjusting the air cylinders, lowering the section to the bottom of the trench and pulling it home, so that the keys could be inserted in the pilot pins, thus locking the sections together (when the tubes would be considered safe and beyond any great risk), took from, say, 8 A. M. until 8 or 9 P. M., after the first two or three sections had been sunk. The lining up of the tubes at the outer end, and the bolting up of the flange connections, could then be commenced the next day. The latter required generally two days, there being about 50 bolts to a joint, or 100 bolts to a section.

A great deal could be said on the subject of placing the exterior concrete around the tubes, but, as this discussion is already too long, the writer will simply refer any one interested to his paper, "Laying Concrete Under Water—Detroit River Tunnel."* It might be stated, however, that a new principle was established at Detroit, with respect to placing concrete under water, namely, that by means of the tremie, properly constructed and operated, concrete may be deposited so that the great mass of it will not come in contact with the water at all, after the first surface of concrete has been formed. This is accomplished by mixing it so wet that the mouth of the tremie at all times is buried in it, thus sealing the end of the pipe and controlling the flow by raising or lowering the tremie in the concrete, and by confining its lateral flow in compartments which are filled one at a time, the concrete all the time seeking its level within the compartment.

For a length of about 1 000 ft. in the middle of the river the bottom of the trench became very soft after it had been excavated, although the digging had been quite hard. It was difficult to assign any reason for this, except the probable existence of several springs in the rock underlying the clay, which, when most of the latter had been removed, would force themselves through the remainder of it, converting it into a puddle. The existence of such springs had been discovered when the borings were made, the water in the casings often rising higher than the river level. To secure a proper foundation for the tubes where this occurred, the wooden sheeting was run down straight, thus increasing the width of the base by about 11 ft. The tremies were then lowered as far as they would go through the soft clay, and, under their own weight (from 7 to 8 tons when filled with concrete), they would generally go down to within 1 or 2 ft. of the rock bottom at a depth of from 85 to 90 ft. below the surface of the river. A little

* *Proceedings*, National Association of Cement Users, Vol. 6, 1910, p. 180; also in *Engineering News*, March 17th, 1910.

Mr. Hoff. extra force was used to put them down as far as possible, and concrete was then deposited until it reached the underside of the diaphragms. In this manner, judging from the number of cubic yards deposited, a series of piers, 6 ft. in diameter, was built up under the tubes, two to a pocket longitudinally of the tunnel, and three rows, one for each tremie; that is, there was one row under the center wall and one under each of the side-walls of the tunnel.

Two methods for unwatering the tubes were considered, namely, by pumps or by air; the latter, after some study, was abandoned. The air pressure required would have had to be at least equal to the hydrostatic pressure at the bottom of the tunnel, or say 30 lb. per sq. in.; this air pressure, of course, would have been uniform at all points inside the tube. The hydrostatic pressure, however, would have varied with the depth; it would have been 20 lb. at the top of the bulkheads and 25 lb. at the center; thus the inside air pressure at these points would have been 10 and 5 lb. in excess of the water pressure outside, which would have tended to blow out the oakum caulking between the timbers. The bulkheads would have had to be designed to resist pressure from the inside as well as the outside. It may be of interest to note that the pumps (8-in. vertical, each having a capacity of 5 000 gal. per min.) were installed in the first section before it was launched, and remained submerged more than 7 months before they were started in order to pump out the first section. Nevertheless, there was not the slightest difficulty in starting them. The unwatering, as a rule, required 3 hours per tube in a section, or 6 hours for two adjoining tubes, when the full-pressure bulkheads were placed in every other section, which was generally the case.

On the subject of the interior lining of concrete, little need be said. This lining was made thicker than necessary, as this was the first time this new method was tried, and the quality of the exterior concrete was unknown when the design was made. The thickness was 20 in., or only 3 in. less than required for the tunnel proper of Designs *A* and *C*. On the other hand, the heavy reinforcement, both circumferential and longitudinal, required in these designs, was unnecessary, and only a few longitudinal rods (spaced about 1½ ft. apart) were placed near the inner surface of the concrete to assist in preventing cracks which might be caused by temperature changes or in the setting up and drying out of the concrete. The lining for Section No. 1 was not completed until 21 months after its unwatering; for the other sections, the time varied from 4 to 12 months, and averaged about 8 months. If necessary in order to save expense, such a tunnel might remain unlined, in which case the steel shell ought to be made a trifle heavier (by, say, ⅛ in.) than required to resist the hydrostatic pressure. Generally, it is preferable to use, say, a 12-in. lining for a tunnel of the size at Detroit.

Mr. Hoff.

One object in developing the adopted design was to reduce the use of divers to a minimum. Diving is expensive work, and frequently unreliable. The only physical labor performed by divers was the temporary attaching of the steel tubes to the grillages in the trench, the bolting up of the joints of the tube sections, and the disconnecting of the air cylinders. In other respects, divers were only used as "eyes to see with," that is, as inspectors to report conditions. The total cost of diving at Detroit was only about one-half of 1% of the total cost of the river tunnel.

At the outset, apprehension was felt by some that trouble might be caused by the trench silting up; such fears were groundless, however. Although part of the trench remained open for almost a year, there was no silting up by material carried in suspension by the river. Section No. 2 was in direct line with the flow of sewage discharged from a large intercepting sewer above the tunnel site. Some of this sewage found its way into the trench, but was easily pumped out. When Section No. 1 was unwatered, it was found that gas from this sewage had been formed inside the tubes and had to be drawn out; some of it remained, however, being pocketed between the outer bulkhead and the semi-bulkhead next to it; this gas exploded when a workman unwittingly struck a match, but no damage was done. In fact, the construction of the whole river section was singularly free from accidents, not a single fatality occurring in connection with the river work—a striking contrast to the general experience when compressed air and shields are used. It may be of interest to note that three of the largest concerns in the country writing Employers' Liability Insurance offered to give protection at about one-third the rate they charged for tunnel work done under compressed air.

One feature of this method of subaqueous tunnel construction which is of great advantage is the small labor cost, as compared with the compressed air and shield method. On the other hand, the aggregate cost of the materials will generally be larger, but not enough to overbalance the labor cost; the result is a stronger and more durable construction, certainly a desirable quality in a subaqueous tunnel. If the bottom is soft, there is nothing to prevent piles being driven to a level below the diaphragms, or using other means for producing a satisfactory foundation.

At Detroit this method effected a saving of about $2 000 000 over a shield-driven tunnel with compressed air, and to this should be added the capitalized saving in annual cost of operation on account of the tunnel being placed some 15 ft. higher, thus reducing by this amount the vertical lift of the tonnage passing through. This distance of 15 ft. meant a saving of 1 000 ft. of approach tunnel on the Canadian side, having a maximum gradient of 1.5%, and 750 ft. on the American side, with a gradient of 2 per cent.

Mr. Hoff. Another great advantage of this method is the fact that for long tunnels work may be started at several points, and tubes may be sunk, working both ways, thus greatly reducing the time of construction.

Before leaving the subject of the subaqueous tunnel, attention must be called to a matter of great importance: The hydrostatic pressure inside the steel tubes must at all times be maintained equal to that outside until the exterior concrete has been placed. This is readily done by keeping the inlet valves open, or keeping the air escapes back of the semi-bulkheads open until the last pocket has been filled with concrete, thus providing direct communication between the water inside and the water outside the tubes. If this precaution is not taken, the risk is run, in unwatering an adjacent section which has been completed, that water may leak through the full-pressure bulkhead between the two sections if not absolutely tight, and thereby expose the steel shell, where not yet encased in concrete, to the full hydrostatic pressure from the outside without a corresponding equalizing pressure inside, thus causing it to fail by deformation. A temporary vacuum might even be produced, thus exposing the shell to the atmospheric pressure as well, in which case it would surely collapse. As soon as the exterior concrete is placed there can be no danger of this, and, in any event, the possibility of such an occurrence can very easily be prevented.

Approach Tunnel.—The Detroit River Tunnel presents the anomaly of having the approach tunnels wholly or in part driven by shields and compressed air and the subaqueous tunnel built without the use of it. This was due to the peculiar plastic quality of the blue clay, which at a depth of 25 to 30 ft. from the surface had a tendency to flow when opened up by shaft or drift, and, as a consequence, developed enormous pressures. Frequently this material has been termed a "swelling" clay, but this is a misnomer—it flows. In the drifts originally driven for the center wall on the Canadian side, the posts of the drift, consisting of 12 by 12-in. maple timbers, 8 ft. long, spaced 2 ft. apart, would break, while the caps, of the same dimensions and length, would hold. Assuming a pressure on the posts per square foot equal to the depth of the drift from the surface multiplied by the weight of the clay per cubic foot (about 140 lb.) a fiber stress of combined bending and compression would be developed amounting to about 7 000 lb. per sq. in., thus producing rupture. To resist this flowing tendency and partly counteract the resulting pressure, compressed air at about 15 lb. pressure was used.

On account of the design of the approach tunnels providing for a center wall, it was necessary to drive practically three tunnels; one for the purpose of constructing the center wall and one for each of the two tracks. It would have been much cheaper, and would have greatly

simplified the operations, if the approach tunnels had been designed as two separate, circular tunnels, one for each track, so that each could have been driven by a circular shield. Mr. Hoff.

After two years of constant effort and sacrifice of health to make this new and untried method of subaqueous tunneling a success, it was a source of the greatest regret to the writer that he was unable to remain with the tunnel and witness its completion; circumstances beyond his control made it necessary for him to retire from the contracting firm in October, 1908, after Section No. 6 had been sunk, and after practically all problems presenting themselves in connection with this work had been solved.

Wilson Sherman Kinnear, M. Am. Soc. C. E. (by letter).—In compliance with Mr. Dunham's wishes, it would have pleased the writer to have described the process of elimination followed by the Advisory Board of Engineers in reaching conclusions as to the best method to be used in the execution of the subaqueous work, under the basic principles established, but it was feared that such an explanation would prove tedious, and form no valuable record of the completed work. Mr. Kinnear.

Several independent or alternative designs were submitted by contractors, based on suggestions made by the Advisory Board, and one of these, in the nature of a modification of Design *B*, was favorably considered by the Board, and might have been used had some disagreement not arisen as to the form of contract. The completed work speaks for Design *A*, but the writer is not prepared to say that Design *B* would not have been equally successful. It is entirely possible that a combination of the two designs might be used to advantage on similar work.

The limit of weight, in air, of a section of tubes, or a partly completed construction, which can be successfully handled, was not reached in the Detroit work, and thicker walls can be connected with ease. It would be difficult to define or establish limiting conditions, but they would be governed largely by depth of water, current velocities, and floating equipment.

The air cylinders were used for efficiency and refinement of work, rather than for economy, and, when combined with the floating equipment described, afforded a control little less than perfect.

Mr. Dunham is wrong in his reading, that in two sections the tubes were filled solidly with concrete. In the east-bound tube at the joint, between Sections 5 and 6, the space between the end bulkheads, some 10 or 12 ft. in width, was filled with concrete from the surface of the water, through a manhole in the top of the tube. The same treatment was accorded the joint between Sections 6 and 7 in the west-bound tube. Thus, a concrete bulkhead was provided in each tube, reinforcing the full-pressure timber bulkheads and insuring complete protection for the unwatering and lining of sections shoreward from the bulk-

Mr. Kinnear.

heads. This action was taken merely as an extra precaution during construction.

The method of unwatering was very simple, and, in the writer's opinion, nothing would have been gained by emptying one tube or section of tubes independent of the contiguous sections, as suggested by Mr. Buerger.

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TRANSACTIONS

Paper No. 1213

A REINFORCED CONCRETE STAND-PIPE.*

By W. W. Clifford, Jun. Am. Soc. C. E.

With Discussion by Messrs. William Fry Scott, William Mueser, W. J. Douglas, Joseph A. Powers, and W. W. Clifford.

The reinforced concrete stand-pipe built at Westerly, R. I., during the summer of 1910 for the town water-works, is of interest because of some methods of construction which were used for the first time, and also because, in appearance and water-tightness, it is one of the most successful ever built.

Westerly is a town of about 9 000 inhabitants. Its water supply is derived from driven wells and distributed by pumping. Prior to 1911 the only reserve supply was contained in a steel stand-pipe having a capacity of 370 000 gal. Additional reserve was thought advisable, and, as the old stand-pipe needed a thorough overhauling, it was decided to build a new one on Quarry Hill, very near the old one. Concrete was chosen as the material, in preference to steel, after the Water Commissioners had examined other concrete stand-pipes in New England.

The general shape of this stand-pipe is shown by Fig. 1, and Plate LIV. The cement seemed to give the concrete a somewhat lighter color than usual, and this was increased by the lime which was added, the result being an almost white concrete. As steel forms were used, its

* Presented at the meeting of November 15th, 1911.

surface was very smooth. The jointing seen in Plate LIV is due to the fact that the forms were not absolutely water-proof at the joints, and the water running out caused a slight burr at the edge of each panel. The finishing tiles of the dome are dark red and glazed, and, forming a marked contrast with the light concrete, give a distinctly pleasing appearance.

The stand-pipe is founded on hardpan, which at this point is only 5 or 6 ft. below the surface. The inside diameter is 40 ft.; the height, from the floor to the overflow, is 70 ft., and from the ground to the top of the ventilator on the dome, is about 88 ft. The thickness of the wall at the floor is 4 ft., tapers to 14 in. at a height of 5 ft., and is of this thickness up to the water line. The wall, for the first 5 ft. above the ground, has an outside diameter of 44 ft. 4 in., then an ornamental moulding reduces it to 42 ft. 4 in., which is constant to the bottom of the triglyphs, 6 ft. below the water line. Just above this there is a fillet 6 in. deep and projecting 4 in. Above the water line there is a cornice 24 in. deep and projecting 30 in. This is surmounted by a parapet wall 4 ft. high. A Guastavino dome of red tile springs from a seat 2 ft. above the water line. Its diameter is 41 ft. and its rise 13 ft.

A steel ladder, 1 ft. wide, of ½ by 1½-in. flats and ¾-in. rounds, is secured by bronze bolts in cast-iron sockets with 1-in. bronze faces, set into the wall at 16-ft. intervals. The rungs are 12 in. from center to center, but this spacing is reduced to 6 in. through the opening in the cornice. The ladder was erected in 16-ft. sections, and the bottom is about 16 ft. from the ground. Over the parapet the the flats are replaced by 1½ by 2½ by ¼-in. angles.

For construction purposes, a frame tower, large enough for a 1-yd. Ransome auto dump bucket, was placed so that it cleared the outside edge of the cornice by about 1 ft. This tower had 6 by 6-in. uprights, and was thoroughly cross-braced. A No. 2 Smith concrete mixer, run by steam, was set in a pit so that the materials could be conveniently dumped into the hopper from the ground. The mixer emptied directly into the bucket, which was run by a hoisting engine. The same boiler furnished steam for mixing, hoisting, and later, also, for pumping water to the top of the wall for washing it. The concrete for the foundation, floor, and base was hoisted about 20 ft. and dropped into a chute; one section of this chute carried the concrete

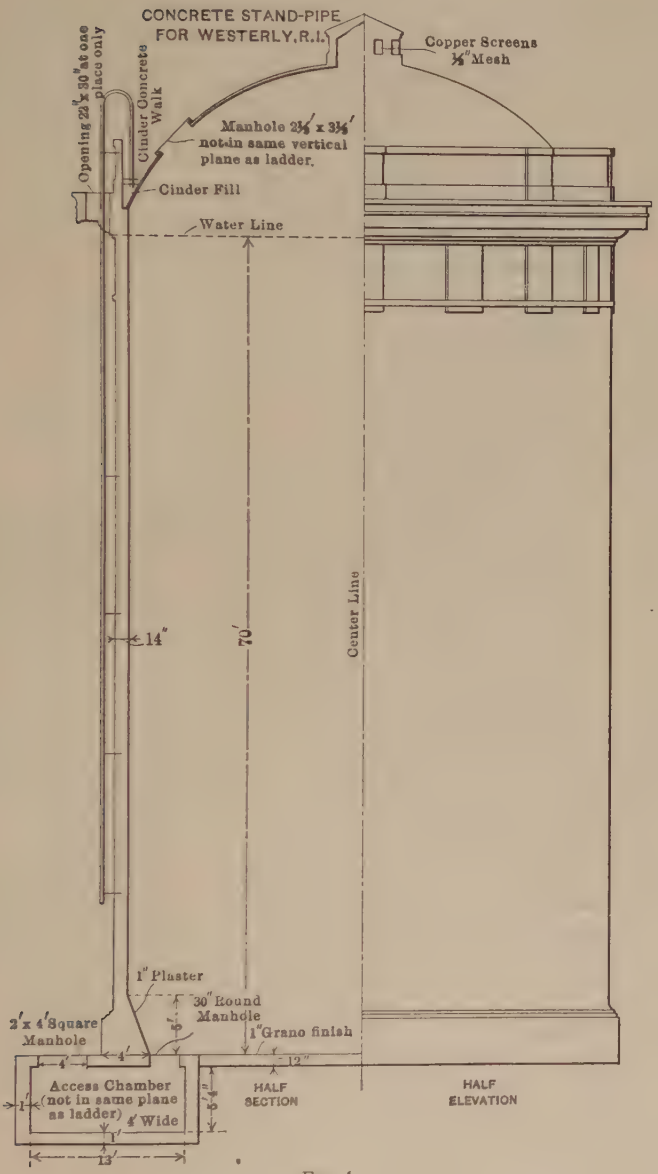
CONCRETE STAND-PIPE
FOR WESTERLY, R.I.
Copper Screens
⅛" Mesh
Opening 22" x 30" at one place only
Cinder Concrete Walk
Manhole 2½' x 3½'
not in same vertical plane as ladder.
Cinder Fill
Water Line
14"
70'
Center Line
1" Plaster
30" Round Manhole
1" Grano finish
2' x 4' Square Manhole
4'
4'
12"
Access Chamber
(not in same plane as ladder)
4' Wide
1'
1'
13'
HALF SECTION
HALF ELEVATION

FIG. 1.

from the tower to the center of the tank, and from there a movable section delivered it in place.

An inside stage, Fig. 2, was used for all but the first few feet of the wall. The floor of this stage was framed of two rings of 8-in. 11¼-lb. channels. These rings were 38 and 28 ft. in diameter, respectively, and were in four sections, bolted together with standard

DIAGRAM OF INSIDE STAGE FOR CONCRETE STAND-PIPE

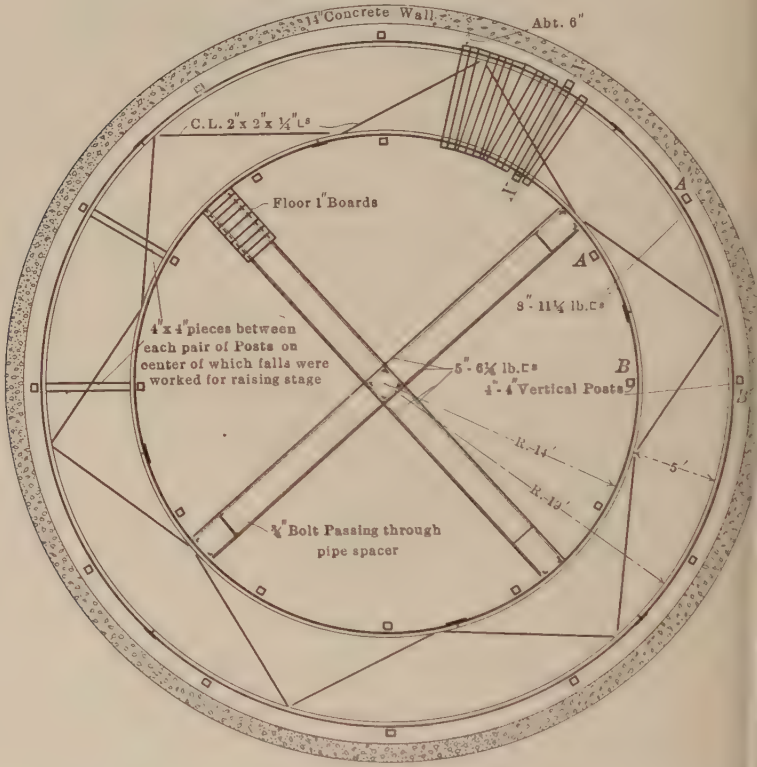

FIG. 2.

splice-plates, the splices being staggered on the two rings. Two bra each consisting of two 6½-in., 5-lb. channels, held parallel and 18 apart by bolts passing through pipe spacers, were placed diametric across the inside of the inner ring at right angles to each ot There was a lattice bracing of single 2 by 2 by ¼-in. angles tween the curved channels. The upper flanges of the curved chan

REINFORCED CONCRETE STAND-PIPE AT WESTERLY, R. I.

were bored every 3 ft. to receive the bolts for the nailing strips. Planks, 2 in. thick and 6 ft. long, were nailed on the curved channels, making a solid floor of this width which cleared the inside of the wall by 6 in., and was very rigid and satisfactory. A floor of boards, 2 ft. wide, ran across one pair of diametrical channels, and on this were stored wheel-barrows, and other tools when not in use. The uprights for this stage were 4 by 4-in. spruce posts, spaced as shown in Fig. 2. Ledges, composed of two 1 by 6-in. boards, were nailed on each pair

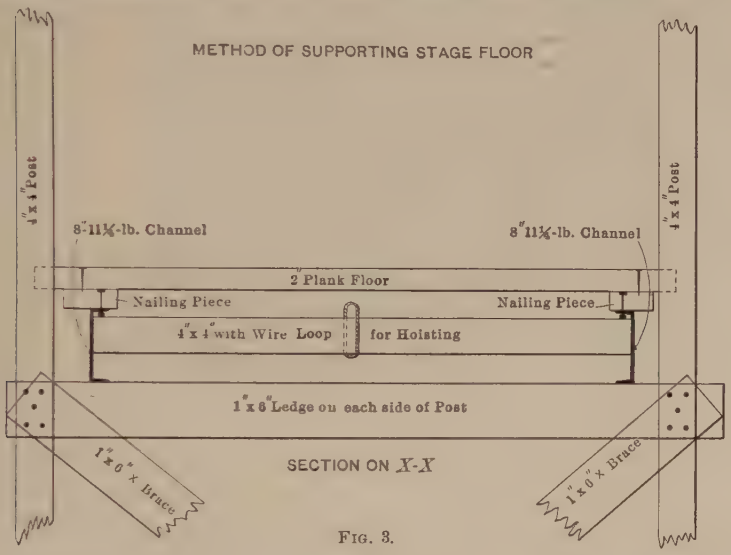

FIG. 3.

of uprights, and the 8-in. channels rested on these. The outside posts went down vertically to the point where the wall commenced to flare in, and there they also bent toward the center and rested on the bottom close to the wall. The bottoms were braced apart by horizontal, radial struts, and the inside posts were all diagonally braced in radial planes down to these struts. Lattice bracing was put in the planes, *AA'*, *BB'*, etc., in 3-ft. panels, or for each lift of the stage, and in the planes, *AB*, *A'B'*, *BC*, *B'C'*, etc., in 6-ft. panels, or for each alternate lift of the stage. All the bracing was of 1-in. spruce boards, 6 in. wide. The posts, in 16-ft. lengths, were kept ahead of the stage, and a 4 by 6-in. cross-head was put across the top of each pair, the inner end at the inner posts, and the other end projecting 2 ft. beyond the outer posts. This cross-head had a wire hook half way between

from the projecting ends of the cross-heads at the top of the staging posts, and men working on these unbolted the lower set of outside forms when they were to be moved up. Ropes were lowered from the stage and hooked into the panels before the last bolts were removed. The forms were then hauled up on the stage, cleaned, greased, and bolted in place on the other set.

The inside forms were handled in a similar way, except that the men operated from the bracing of the inside stage. These forms were a little too light, and caused some trouble by springing out of shape. This made the constant use of drift-pins necessary, to line up the bolt holes. The bolts were subjected to such rough treatment that they could be used on the average only twice. Toward the latter part of the work short boards were nailed on the floor of the stage, in 12 or 15 places around the circumference, to hold the inside form out to the proper radius. Spreaders and wire ties were then put between the tops of the forms at these places. The bottom was held by the form already filled. This method held the forms firmly in place, and took very little time, not more than an hour for three men.

Pieces of 2-in. plank, cut to the arc of the outside circumference, were bolted under the top flange of the twentieth form, and supported wooden forms for the lower ends of the triglyphs and the fillet. Both steel forms were set up on the fillet, with spacers between the panels to spread them to the outside diameter of the triglyphs. Through holes bored in the steel forms nailing strips were bolted, and vertical lagging was nailed to these, to make boxes for the indentions between triglyphs. The pieces of plank which had supported the fillet forms were again bolted under the top flange of the forms, and supported wooden forms for the short section of wall between the top of the triglyphs and the bottom of the cornice.

The plans called for overflow holes, 12 in. wide and 3 in. deep, spaced 45° apart around the circumference, but as these openings provided the only opportunity to support the cornice forms, they were spaced 11¼° apart, and the extra ones were filled up afterward. One day's pouring was ended at the top of these openings, and flat concrete blocks were placed over them and incorporated in the wall, in order to obviate the difficulty of removing forms from such small openings. Pieces of 4 by 4-in., 6 ft. long, were cut down to 3 by 4 in. for 18 in. at one end, and two were put in each overflow hole, making

as great an angle as possible with each other. The outer ends of these brackets were braced down to the fillet 6 ft. below. On these 64 brackets the cornice forms were built, partly of lumber sawed in the yard and partly of plaster of Paris on metal lath which was used so that the concrete would not be damaged by the swelling of the wood when wet. The plaster was composed of equal parts of plaster of Paris and Limoid. It was placed and shaped by trowel and template, and smoothed with a 4-in. paint brush and water while still green.

The outside base forms, with boxes built inside to give the proper shape, were used in building the parapet. The inside steel forms were used from the top of the base to the top of the parapet, spreaders being put between the panels above the dome seat. For carrying up the center, short vertical planks were bolted into four sockets inside the wall. These were set at the same height and equally spaced. To the tops of these planks two wires were fastened so as to stretch diametrically across the tank. By revolving the planks slightly about the bolts, the wires were adjusted so that their intersection was exactly at the center. This arrangement was put in every 15 or 20 ft., for the tank was filled as the work progressed.

The centers for the dome were supported by the inside stage, which was left in for that purpose, and later all the long pieces were taken out through a manhole in the dome by a gin pole placed on the cornice. The smaller pieces were taken out through the access chamber.

The main reinforcement was of plain, round, mild-steel bars, 1½ and 1⅝ in. in diameter. These bars were 71 and 69 ft. long, so that two formed a complete ring, allowing 40 diameters for lap. The bars were bent on the ground, by from 4 to 6 men, blocks being set by trial to give the necessary curvature. They were then hammered to set. They were hoisted by two No. 0 Parker derricks, and, in addition, it was found necessary to use one of the swinging stage falls on the middle of the bar. On arriving at the top of the wall, they were taken to their position by hand. These hoops were supported by twelve equally spaced 1½-in. vertical iron pipes, resting on standard 6-in. flanges on the floor. These pipes were in 3-ft. sections, corresponding to the height of a form. The sections were joined by ordinary couplings, and erected as the work progressed. Braces of 2 by 2 by ¼-in. angle iron connected these pipes while the steel was

being put on. These were bolted into special castings screwed into the couplings on top of the last erected sections of pipe, and were removed when the steel was in place and another section of pipe was to be erected. Radial ¼-in. holes were bored through these pipes at the proper intervals for the rods, and through these holes were put ¼-in. hooks, in which the rods were placed. The hooks were then hammered flat against the pipe or around the bar. In some cases it was found necessary to use some No. 10 wire, as well as the hooks, to fasten the rods to the pipes. The pipes were filled with grout as the work progressed. The joints were staggered so that about one in twelve came in the same vertical plane. Two Crosby clips, such as are commonly used for guy wire, were also used on each joint.

In the floor the secondary reinforcement is composed of ¼-in. square rods, 6 in. from center to center both ways. The rods are 1 in. below the surface, and are bent up 4 ft. into the wall. Around the manhole and roof of the access chamber, ⅝-in. round and 1¼-in. square rods are used. Just under the dome seat there are ¼-in. square rods, set horizontally, and ⅝-in. round rods, 4 ft. from center to center, are bent out into the cornice. Round ⅝-in. rods, 7 ft. long and 2 ft. from center to center, run vertically up into the parapet.

The proportions for the concrete were decided after numerous carefully made experiments, for dependence was placed on the density of the concrete to make it impervious.* The materials available were crushed granite and several bank sands, all of good quality. Vulcanite cement was used. The granite (gray Westerly) was a satisfactory larger aggregate. The sand chosen was that which contained the least vegetable matter and showed the greatest strength when compared with a similar mixture of mortar made with standard Ottawa sand. Samples of sand and of stone were sifted through a set of sieves, in order to ascertain the percentage by weight retained on each sieve. From these data curves were plotted, using the percentages passing through the sieves as ordinates and the sizes of the sieves as abscissas. The sand (Curve *A*, Fig. 4) ranged from 1½-in. gravel down to grains passing through a 100-mesh sieve. The stone (Curve *B*, Fig. 4), which was supposed to be crusher-run below 1½-in. with

* The methods of mechanical analysis and the volumetric tests used were those discussed in Taylor and Thompson's "Treatise on Concrete: Plain and Reinforced."

the dust screened out, ranged from that size down to a little very fine dust which was carried over the screen by the larger pieces. Then a curve was plotted which, according to the experiments by Messrs. Taylor and Thompson, represented a mixture of sand and broken stone of maximum density. This is shown on Fig. 5. The mixture of the available sand and stone which gave a curve nearest to this ideal was calculated (Curve *C*, Fig. 5). From the calculations and the shape of the original curves, it was seen that the materials could not be combined to produce a curve in which the maximum deviation from the ideal curve was less than 6 or 7%, that is, they could not be combined to form a curve of this shape, but it was plainly seen that if the finer stone or the coarser sand was removed, a combination curve within 2 or 3% of the ideal could be obtained.

All the experiments seemed to show that the maximum density curve would be a smooth one of approximately the ideal shape, as the different materials gave results differing only in the location of the curve. Finer sand was obtainable from the same bank, but its quality was not as good as the coarser, therefore it was decided to re-screen the stone over a ½-in. screen. This new stone, when combined with the coarser sand, gave a mixture curve very close to that of the ideal (Curve *B*, Fig. 5).

A 50-lb. sample of this combination, and a mixture having a curve which was exactly like the ideal at every screen, were mixed and their volumes measured by taking the distance from the top of a cylindrical can to the surface of the concrete after the sample had been thoroughly tamped in. These two combinations showed the same density, within the limits of error of the experiment, but were distinctly and unworkably lacking in mortar. Proportions varying from this slightly, in both directions, were tried, in order to obtain one having a maximum density. This was finally found to be a mixture of equal parts of sand and stone, the curve for which averaged 6 or 7% higher than the ideal (Curve *D*, Fig. 5). The reason for the variation of the maximum density curve from the ideal may have been because some of the larger aggregate was gravel (the coarser particles of sand), this being approximately one-third, if ¼ in. is considered as the limit between sand and stone. Another possible reason was the shape of the stone, as the granite crushed into flat and finger-shaped pieces, rather than into the regular tetrahedrons of trap.

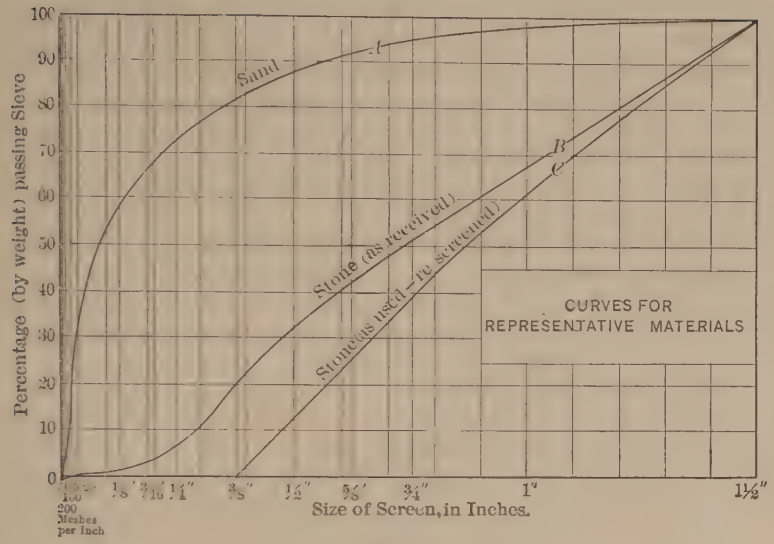
Percentage (by weight) passing Sieve
Sand
Stone (as received)
Stone as used—re-screened
A
B
C
CURVES FOR
REPRESENTATIVE MATERIALS
Size of Screen, in Inches.
1½"

FIG. 4.

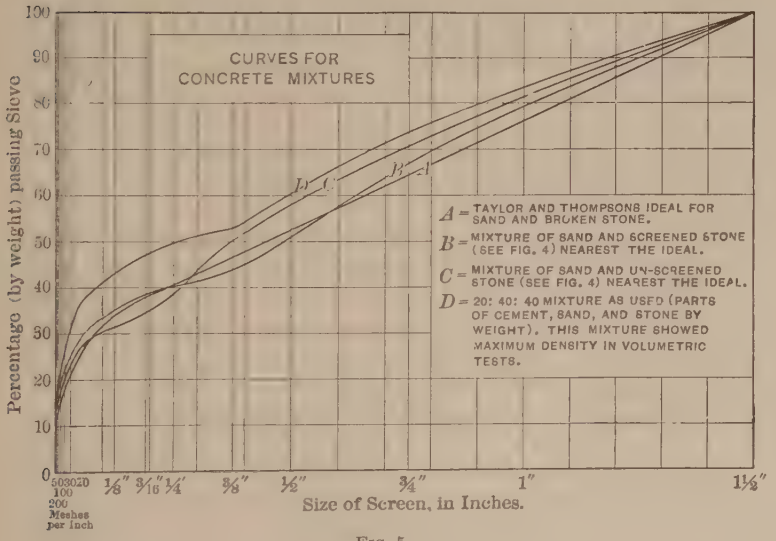
CURVES FOR
CONCRETE MIXTURES
Percentage (by weight) passing Sieve
A = TAYLOR AND THOMPSONS IDEAL FOR SAND AND BROKEN STONE.
B = MIXTURE OF SAND AND SCREENED STONE (SEE FIG. 4) NEAREST THE IDEAL.
C = MIXTURE OF SAND AND UN-SCREENED STONE (SEE FIG. 4) NEAREST THE IDEAL.
D = 20: 40: 40 MIXTURE AS USED (PARTS OF CEMENT, SAND, AND STONE BY WEIGHT). THIS MIXTURE SHOWED MAXIMUM DENSITY IN VOLUMETRIC TESTS.
Size of Screen, in Inches.
1½"

FIG. 5.

The proportion of cement was determined arbitrarily from the fact that it was thought best by the engineer not to get more than 12 cu. ft. of concrete from a barrel of cement, in order to obtain the required strength and density for a 14-in. wall under a 70-ft. head. A 20:40:40 mixture by weight, of cement, sand, and stone (sand being considered as it was delivered on the job, and containing some gravel, which should properly be classed with the larger aggregate) seemed to satisfy all the conditions. This made about 11½ cu. ft. of concrete per barrel of cement, and showed a maximum density in volumetric tests (Curve *D*, Fig. 5). Samples of the materials were analyzed every day or two, and their curves plotted. From these curves the necessity for any change in the proportions could be noticed. This mixture approximated a 1:1½:3 more nearly than any other standard mixture, and was used with little variation from the bottom of the floor to the overflow line. Above this, approximately, a 1:2:4 mixture was used. In the foundations stone was used up to 2 in. in size, in a mixture which was approximately 1:3:6.

The mechanical analyses of sand and stone were made on samples weighing 6¼ lb. (100 oz.), the scales weighing to ounces and quarters. Samples of sand of this size were handled conveniently, and were very satisfactory. With the stone, however, there was great difficulty in obtaining a truly representative sample so small, even by quartering, hence some of the stone tests showed rather erratic results.

The screens used were: 1½, 1, ¾, ½, ⅜, ¼, $\frac{3}{16}$, and ⅛ in., and 8, 20, 30, 50, 100, and 200 meshes per linear inch. The volumetric tests were made with a 50-lb. sample mixed with a shovel on a 3 by 5-ft. galvanized-iron pan having the edges turned up 1½ in. The sand and cement were mixed dry, then water was added, and then the stone was thoroughly incorporated, following one of the customary methods of hand mixing. In the earlier experiments, the samples were measured in a galvanized-iron can, 10 in. in diameter and 15 in. deep. Later, a piece of 6-in. wrought-iron pipe with a removable flat plate on the bottom was used. The latter gave greater accuracy, as the ratio of depth to cross-section for an equal quantity was greater. The relative volume was determined by measuring the distance from the surface of the sample to the top, as both the cans were truly cylindrical. The samples were put into the receptacle with a bricklayer's trowel, and, as placed, were thoroughly tamped with a 2 by 4-in.

rammer until water rose to the surface. Tests of the voids in the sands and stone showed that the volume of cement was 10% in excess of the voids in the sand, and the volume of mortar was more than 10% in excess of the voids in the stone, as was required by the specifications.

The only water-proofing agent used was Limoid (a patented form of hydrated lime) to the extent of 5% by weight of the cement. Some tests made by the writer seemed to indicate that the addition of lime to cement increased its volume by approximately the volume of the lime added. As the cement was more than enough to fill the voids in the sand, and the mortar to fill the voids in the stone, it is difficult to see wherein lay the advantage of the lime.

The joints were given special care, and the separate day's work seemed to be completely bonded. The top surface of the concrete was thoroughly cleaned each night after it had taken its initial set, and all the laitance was removed, leaving clean sand and stone surfaces exposed. At the beginning of each pouring the surface was covered with grout, and a 1-in. layer of 1:3 mortar was put in before the concrete. The first joint above the floor could not be cleaned before final set, because the floor finish was fresh, and water could not be run over it, so it was found necessary to clean the joint with muriatic acid, and even this was not sufficient for some parts, and they had to be picked.

The floor had a 1-in. granolithic finish, and this was carried up as a plaster coat to the top of the inside bevel. The outside surface was rubbed with carborundum and then painted with a grout wash, except on the base, which, from the ground up to a level 2 in. below the top of the moulding, was picked.

Several porous spots showed dampness on the outside when the tank was first filled. These spots were quite wet in the morning, but the sun dried the most of them in the course of the day. They were treated by forcing grout into them. A cavity was picked in the wall for 7 or 8 in. at the point which seemed to be the source of the leakage, and a $\frac{3}{4}$-in. nipple, 6 in. long, was bedded in this with 1:2 mortar, the inner end being kept clear by filling stone around it. Grout was then forced in, under a pressure of about 100 lb. per sq. in., furnished by a tank of carbon dioxide gas, such as is used at soda fountains. This eliminated these leaks. Another leak, which appeared

along the top of the moulding on the outside, showed as a horizontal crack running nearly half way around the circumference, and varying in amount of leakage. As this was too big to repair by grouting, it was decided to water-proof it from the inside. A paraffin treatment was tried on the wall for 8 ft. above the floor. The wall was heated with charcoal furnaces, and hot paraffin was applied; then the wall was reheated, thus forcing the paraffin into the pores. Several coats of this were applied, but they have not proved sufficiently elastic to keep the wall tight under stress.

Apparently this leak is a stress strain, due to the unequal expansion of the 14-in. wall and the thickened base. The original design had been changed by bringing the top of the inside bevel above the outside moulding to prevent this very difficulty, but without entire success. This is the only structural defect which has been brought to light, thus far, and, in the design of concrete tanks under high pressure, is one of the greatest difficulties to overcome. In addition to the differential expansion, there is also the change of stress from the compression on the bottom to the tension on the wall. If there is no projecting base, the source of the difficulty is simply lowered to the junction of the floor and wall. All the porous spots were on the south side, and most of them were also in the part of the wall nearest to the construction tower, where drippings from the chute fell into the wall forms.

The force engaged was composed of about 25 men: a superintendent, an engineer, 8 carpenters, 14 laborers and an engineman. The carpenters did the work on both the wooden and steel forms, and most of the work on the reinforcing. The laborers did the concrete work, screened the stone, unloaded materials, and acted generally as helpers.

The lower section of the wall and the floor were put in on June 15th in 20 hours of continuous work. Each of the first few sections above the floor took 2 or 3 days. When well started, however, a 3-ft. section was poured in a day. This meant placing the steel and moving up the forms, in which all the men were used, the laborers as helpers, cleaning and greasing the forms, etc., then the stage was raised, usually about noon, and the concrete was poured in the afternoon. It was allowed to set for a few hours while the men were clearing up and getting ready for the next day's work, then in the early evening the concrete foreman and three or four laborers cleaned

the top surface. The concrete was finished and ready for the dome 10 weeks after the floor was put in.

The cost of the work is given in Table 1. In considering these costs note should be taken of the fact that certain parts of the work were done under pressure; namely, those parts for which the whole work waited. Other parts were done in a more leisurely manner, owing to the fact that men cannot work continuously at their maximum speed. For example: In the morning the first thing done was to place the steel, secondly, to raise the forms, and thirdly, to raise the stage. All these had to be done before concreting could begin. This work was done by the carpenters working at maxmium speed, some of the laborers acting as helpers. During this time the other laborers were screening stone, washing down the walls, etc., under no great pressure. Later in the afternoon the laborers were working at top speed on the concrete, while the carpenters were placing the necessary bracing on the staging, and getting ready for the next day's work, all at a less forced speed. Consequently, the labor costs for reinforcement, forms, and concrete, show up much better than those for staging, screening stone, finishing the wall, etc.

The materials were delivered by a granite company, on a side track about 100 ft. from the site of the work, for 35 cents a ton, the additional costs being for the labor of unloading and carrying the materials to the site of the work. The steel was carried by hand, and the cement in wheel-barrows. The stone and sand were delivered in piles beside the mixer by carts.

The following prices were paid for labor:

Foreman carpenter	48	cents per hour.
Carpenters	43¾ and 45	" " "
Carpenter's helper	35	" " "
Engineman	35	" " "
Labor foreman	50	" " "
Laborers	22½ and 25	" " "

The following prices were paid for materials:

Cement	$1.52	per bbl. (less 30 cents for bags returned).
Sand, delivered at site	1.15	" yd.
Stone " " "	1.07	" "
Limoid	1.00	" bag (100 lb.).
Plaster of Paris	2.00	" bbl.
Steel	38.00	" ton, plus the freight.

TABLE 1.—Cost of Reinforced Concrete Stand-Pipe at Westerly, R. I.

Items	Quantities.	Cost per Cubic Yard. Labor.	Plant and stage.	Cement and Limoid.	Aggregate.	Total unit cost.	Total cost.
Concrete:							
In foundations	337 cu. yd.	$0.90	$1.44	$1.43	$2.09	$5.86	$1 975
" floor	119 " "	1.52	1.44	3.40	1.56	7.92	942
" walls and cornice	423 " "	1.74	2.14	3.34	1.56	8.78	3 714
" parapet	13 " "	1.00	2.14	2.53	1.56	7.23	94
" walk around roof	.....	.....	.....	.....	.....	.....	60
Finish:		Cost per 100 Sq. Ft. Labor.	Lumber.	Cement and Limoid.	Aggregate.		
Rubbing and wash on walls	20 866 sq. ft.	$1.14	.....	$0.03	.....	1.17	244
Picking on plinth	573 " "	14.14	.....	.....	.....	14.14	81
Plaster on plinth	132 lin. ft.	0.46	$0.76	0.04	.....	1.26	166
Picking angle for plaster	.....	.....	.....	.....	.....	.....	33
Granolithic on floor and plastering angle	1 767 sq. ft.	5.85	.....	4.43	$0.15	10.43	184
Granolithic on cornice	400 " "	2.25	.....	1.00	0.15	3.40	14
Granolithic on outside walk	343 " "	1.84	.....	3.60	0.15	5.59	19
Forms:		Cost per 100 Sq. Ft. Labor.	Plant and stage.	Material.			
Foundation	450 sq. ft.	$18.29	.....	$3.40	.....	21.69	98
Plinth	1 249 " "	12.50	.....	2.20	.....	14.70	184
Walls	16 733 " "	4.40	$3.36	5.46	.....	13.22	2 212
Walls, triglyphs	1 208 " "	18.00	3.36	20.24	.....	41.60	502
Floor	247 " "	5.08	.....	2.20	.....	7.28	18
Cornice	1 163 " "	38.32	3.36	13.58	.....	55.26	643
Parapet	1 225 " "	13.63	.....	8.41	.....	22.04	269
					Carried forward	.....	$11 452

TABLE 1.—(*Continued.*)

					Brought forward.....	$11 452
STEEL REINFORCEMENT:						
		Cost per Ton.				
		Labor.	Stage.	Material.		
Rods	68 tons	$9.73	$4.50	$99.00	$53.23	3 619
Crosby clips						171
Pipe standards for steel	852 lin. ft. at 20 cents					169
Hook spacers	2 200 at 1¼ cents					28
OFFICE AND GENERAL:						
Telephone, stationery, traveling, etc						200
Board						320
Travel, and superintendence (at beginning of work)						56
Watchman, and lights						50
Delay (waiting for lost cars of steel)						104
Clearing site, and grading						120
Setting pipes, manhole cover, and tablet						17
Miscellaneous extra work, and pipe-laying						196
Material for steel ladder, $90.50; setting and painting ladder, $44.25						135
Guastavino tile dome						1 200
Trap door in dome						9
Water-proofing (test blocks)						10
Bond (surety)						105
					Total.....	$17 961

In Table 1, the cost of the stage is divided between concrete, forms, and steel, in the proportions of ¼, ½, and ¼. In the labor costs for the wall steel, about one-third is charged to bending and two-thirds to placing. In the secondary reinforcement, the cost of bending was a negligible quantity.

The capacity of the stand-pipe is 660 000 gal., and its cost was about 2¾ cents per gal. It was designed and built by the Aberthaw Construction Company, of Boston, Samuel M. Gray, M. Am. Soc. C. E., being Consulting Engineer for the Water-works.

DISCUSSION

Mr. Scott. WILLIAM FRY SCOTT, Assoc. M. AM. SOC. C. E. (by letter).—There are two points in this paper, which, if elaborated, would make it much more interesting, namely, to give the actual spacing of the circumferential rods and the construction of the Guastavino dome and lantern, especially its thickness and weight.

Fig. 1 would be improved by having a section showing the relation of the foundation to the superstructure; as given in the paper, it is misleading.

Mr. Mueser. WILLIAM MUESER, M. AM. SOC. C. E. (by letter).—Two statements in this paper are of peculiar interest to the writer: First that this reinforced concrete stand-pipe is, in water-tightness, one of the most successful ever built; and, secondly, the description of the leak in the wall near the base, consisting of a horizontal crack running nearly half way around the circumference, together with the various unsuccessful efforts made to stop this leak, and the author's assurance that it is the only structural defect.

All through the paper evidence is to be found of first-class workmanship and of great care in the selection of materials, and yet there was this horizontal crack, notwithstanding which the author claims that this structure is unusually successful, as far as water-tightness is concerned. The writer believes that this claim is justified, but only because all reinforced concrete tanks and stand-pipes of magnitude leak, some more than others. All have horizontal cracks near the bottom, and all efforts to prevent or close these cracks have been unsuccessful. There is a well-founded and general suspicion that it is impossible to make such structures water-tight, and, if they are of any magnitude, contractors are not willing to guarantee their water-tightness. There is no doubt that in many cases, as in the present one, extreme care has been taken to prevent the leaks and cracks which always appear in the lower part of the wall, but without satisfactory results. These and other facts would seem to bear out the writer's conclusion that the trouble with such tanks is not due to the workmanship but to the design, and, in one particular, there should be a radical change, namely, there should be absolute disconnection between the bottom of the tank and the circular wall. This wall should rest on the bottom in such a way as to allow it to expand and contract readily under changing conditions of water pressure. It seems to the writer that the precedent of tying bottom and wall together has been followed too closely in designing stand-pipes and large water tanks, the same method being used in the construction of vats and small tanks, where enough material can always be used, or, if necessary, wasted, to keep the vessel from breaking or leaking. As a matter of fact, a rein-

forced concrete tank should be considered as consisting of two elements with two radically different functions: The bottom of the tank supports the weight of the water, the pressure being vertical; the wall has to take care of the horizontal pressure. As all materials are elastic, the wall cannot do its work unless it is allowed to expand, that is, to increase its diameter. There seems to be no good reason for preventing this expansion artificially by tying the wall to the bottom, thereby creating secondary stresses of almost unknown quantity, which the designer, after creating them, tries to take care of by thickening the wall at the base or by extra reinforcement. Mr. Mueser.

If the circular wall is placed on the bottom, and is separated from it by a layer of some anti-friction material, all that is necessary is to water-proof the joint between these two elements—a matter of simple detail. The writer has developed a design along these lines, and this has been adopted for a 1 000 000-gal. tank, 60 ft. in diameter and 48 ft. high, to be built at Sanderson, Tex., by the Galveston, Houston and San Antonio Railway Company. This radical departure from present practice may be of interest.

W. J. Douglas, M. Am. Soc. C. E. (by letter).—The writer is very much interested in this paper because reinforced concrete tanks generally leak and because, in spite of that fact, there is a constantly increasing demand for them. The author has devoted considerable space to the water-tightness of the concrete, resulting from the careful selection and emplacement of materials. He then calls attention to a circumferential crack in the wall of the tank, which is apparently the fault of the design. Mr. Douglas.

The writer has given this matter considerable thought for some time, and, having had some experience, has reached the opinion that most leakages in similar tanks are the fault of design, resulting from:

1.—Stressing the steel too high;
2.—Using walls which are too thin;
3.—Having the steel so bunched together as to prevent its proper jacketing by the concrete.

In regard to the first cause of leakage, it should be borne in mind that concrete tends to crack when setting, and this tendency is augmented by a fall in temperature. Further, if the concrete has not cracked as the result of shrinkage caused by setting and temperature, it must evidently crack when the tank is filled with water (based on all existing data as to the extensibility of concrete), when the steel is stressed to 4 000 or 5 000 lb. per sq. in. These cracks may be negligible, and may not leak at all, or may silt up readily. In order to keep them small, the percentage of circumferential steel should be high, and, under full head, should not be stressed more than 12 000 lb. per sq. in., and, in the case of surge tanks for hydro-electric plants,

Mr. Douglas.

the steel in the upper portions, where the stress varies at frequent intervals from zero to the maximum, should have a still lower stress, say from 8 000 to 10 000 lb. per sq. in.

The quantity of diametral steel in the bottom slab of the tank should also be large, otherwise leakage cracks will form between the base and the cylindrical wall. There is no theory which may be satisfactorily applied to ascertain the quantity of steel, but the writer suggests that this steel should be of sufficient strength to carry the full bursting pressure at that point, independent of the tank hooping, the stress not exceeding 18 000 or 20 000 lb. per sq. in. It is also evident that a leakage crack tends to form between the base and the wall, not only at the base, but for several feet above, and, therefore, this bottom steel should be carried up in the wall for a distance of 3 or 4 ft.

It does not appear advisable to depend on surface adhesion to transmit tension at the lap of adjacent bars. No one thinks of building reinforced concrete beams, or girders, with the steel lapped at the middle, where the stress in the steel is the maximum, and certainly there is no reason for considering similar laps sufficient in a reinforced concrete tank. The writer's firm determined by test that ¾-in. rods fastened together with two Crosby clips, tightened by a 24-in. wrench, would not slip under tension below the elastic limit of medium steel, and it is thought that this very efficient method, or one equal thereto, should be used for connecting the laps of all reinforced concrete tank hooping. At least one tank failure has been directly attributed to the inefficiency of a lap of about thirty diameters. When a tank is to be filled while the concrete is still green, the concrete adhesion, of course, is particularly low.

In regard to the use of too thin walls: It appears that not a few designers have tried to see just how thin a wall of reinforced concrete could be built and yet hold water. Tanks have been built from 50 to 60 ft. high, 7 or 8 in. thick at the base and 4 or 5 in. at the top, and they hold water with only slight seepage. This seems to prove conclusively that remarkable things can be done with concrete, but such design certainly does not offer good precedent. The writer believes that all such tanks leak, and it seems only logical that they should.

With thin walls, the cost of construction is increased, in spite of the fact that a smaller quantity of material is used; and encasing the steel is made almost impossible. The writer is of the opinion that a tank with 15-in. walls which do not taper would be cheaper than one of equal diameter and height with walls tapering from 8 to 5 in. It is evident that the leakage in the former, even with the same quantity of steel, would be materially less; and further, in northern latitudes, ice formation within the tank would be negligible, without going to the expense of housing the tank.

Mr. Douglas.

In all water tanks, and in surge tanks particularly, the formation of ice on the inside of the walls of the tank is important. Such formation reduces the available quantity of water very rapidly, as the volume varies with the square of the diameter. Further, ice formed on the surface of the water may cause serious water hammer, against which the tank is designed to provide. Such a condition might be brought about by the water lowering several feet, if the ice is of sufficient thickness to bridge between the walls. Under the ice a vacuum would be formed and would remain until the surface ice had broken. As one of the greatest advantages of reinforced concrete over steel tanks is the prevention of ice formation without housing, it seems highly important to make the walls so thick that freezing cannot take place.

The lateral pressure of ice in the upper part of a tank may also cause serious pressures, and these can be best guarded against by preventing its formation.

The third cause of leakage is self-evident. The old inefficient gravity mixer, which was in reality a "gravity separator," often consisted of a series of bars through which the concrete ingredients were filtered. The circumferential bars of the tank are particularly effective in separating the ingredients, especially when these bars are in two rows, therefore, extreme care must be taken to get compact concrete and to jacket the reinforcing bars closely. It is obvious that, when the wall is thin and the tank high, the bottom steel becomes too closely bunched for good work.

In regard to concrete tanks in general, it appears quite probable that their number will rapidly increase. They are not much more expensive than steel ones, and are cheaper than housed steel tanks, and, under the most favorable circumstances, are even cheaper than unhoused ones.

Concrete tanks of reasonable thickness may be made tight and free from ice in even the most severe weather, and, of course, require little attention and practically no maintenance. As to the proportions of the concrete ingredients to be used in tank work, it is never well to take laboratory methods into the field. Granulometric analyses are indicative, and may aid in the selection of materials, but the design should be made so that hair-splitting field methods are not essential to successful construction. Mr. Clifford states that the cement was 10% in excess of the voids in the sand, and the mortar 10% in excess of the voids in the stone. This seems to be an excellent mixture, and, if the proportions of the ingredients are worked out on this simple principle of voids, an equally impervious concrete will be obtained.

The value of applying water-proofing materials to porous concrete or cracked concrete is not questioned, but the application of such material to a tank which changes its diameter with every level of the

Mr. Douglas. water is of doubtful value. Some contractors seem to think that they can get out of trouble at the eleventh hour by water-proofing the inside of the tank, or that they can do poor construction work provided they have some water-proofing compound in their mixture. The only assumption to make in the design and construction of a tank is that neither water-proofing ingredients nor water-proofing applications after construction are going to help materially a bad design or careless construction.

Mr. Powers. JOSEPH A. POWERS, M. AM. SOC. C. E.—In some features stand-pipes for conveying water to water-wheels are similar to the structure described by Mr. Clifford. The speaker, having to design and construct such a stand-pipe or penstock, studied the matter carefully, and came to the conclusion that he must depend on tensile strength in the concrete or put in a sufficient quantity of reinforcing steel to carry all the strain. Any attempt to make the two work together would be like the attempt of an architect who recently endeavored to convince the speaker of the feasibility of making a span of 26 ft. with two 4 by 14-in. sticks reinforced with a steel truss rod. He admitted that he expected to have to screw up the turnbuckle on the rod from time to time, but demonstrated to his own satisfaction that the strength of the sticks, as beams, should be added to the strength of the rod as a truss. The speaker did not agree with him, and does not think it safe to trust concrete in tension for any part of the strength required in a reservoir such as Mr. Clifford describes. As much steel is required to make the tank of safe strength as to make it entirely of steel. Of course, the riveted joints are a source of weakness, but the speaker would rather trust such joints than the lap of the steel bars held by the concrete.

After calculating the cost of a reinforced concrete stand-pipe 12 or 14 ft. in diameter, the speaker concluded that he would use steel, because he did not believe that concrete was ever intended to be used in tension, or would ever be successful under such conditions. The lesson to be learned from this paper is that the use of concrete for a water-works stand-pipe should be avoided, because of the work required in trying to make this structure water-tight and safe. Attempts to cure defects in concrete construction by the use of a mason's trowel, or by injecting any material into the voids, will only result, at best, in a patch-work job. In other words the speaker believes it to be a good conclusion to leave concrete alone and utilize a material which is just as cheap, or cheaper, and is perfectly reliable. This may seem to be somewhat iconoclastic, but the speaker believes that concrete will not work in harmony with steel in such structures.

There is no doubt about the temptation to use concrete for this purpose. Architecturally, it appeals to any one, and the various Boards

of Public Works and Boards of Water-Works who are seeking something which will suit them are more attracted by the appearance of a well-designed and well-built water tank of concrete than by a steel structure. Mr. Powers.

W. W. CLIFFORD, JUN. AM. SOC. C. E. (by letter).—Mr. Mueser's idea of a slip-joint at the bottom of the wall is very interesting, and the writer would be glad to learn of its effectiveness in practice. Mr. Clifford.

Mr. Powers' statement that the steel reinforcement should be sufficient to carry all the tension without help from the concrete, is hardly open to argument. The steel in the Westerly stand-pipe was designed in accordance with that idea. His reference to the possible lack of dependence to be placed on the lap of steel bars, seems to indicate that he did not notice the statement that two Crosby clips were used on each joint. His conclusion that "the use of concrete for a water-works stand-pipe should be avoided," is not warranted. Lack of entire success in the use of a comparatively new material is not proof that it should never be used. There was a time when steel was not considered "perfectly reliable." In the last paragraph of his discussion, Mr. Powers gives a good and sufficient reason for endeavoring to perfect concrete stand-pipes.

The writer agrees with Mr. Douglas that the steel in the walls should be sufficient to carry all the tension at a low unit stress. He fails to see, however, that running the floor steel up into the wall will settle the difficulty of making tight the joint between the floor and the wall.

The writer would substitute "porosity of concrete" in place of "using walls which are too thin," which Mr. Douglas gives as a reason for leaks. That a 15-in. wall may be as cheap or cheaper to construct than one tapering from 8 to 5 in., is reasonable; but even a 15-in. wall must be of dense concrete. Any one who has studied the proportioning of concrete will realize that mixtures may vary greatly in density and strength, and yet have "the cement 10% in excess of the voids in the sand and the mortar 10% in excess of the voids in the stone." The writer considers that the "laboratory methods" were not too "hair-splitting" for use "in the field," on work of this character.

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TRANSACTIONS

Paper No. 1214

RETROGRESSION IN THE TENSILE STRENGTH OF CEMENT.*

By J. M. O'Hara, Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. Maurice G. Parsons, Clifford Richardson, R. S. Greenman, E. P. Goodrich, E. D. Knap, and J. M. O'Hara.

The Standard Specifications for Portland Cement, adopted by the American Society for Testing Materials, require the minimum tensile strength for briquettes of neat cement, 1 sq. in. in cross-section, to be as follows, and to show no retrogression in strength between 7 and 28 days:

Age.	Strength.
24 hours in moist air	175 lb.
7 days (1 day in moist air, 6 days in water)	500 "
28 " (1 day in moist air, 27 days in water)	600 "

The question the writer proposes for consideration and discussion in this paper is whether a cement which shows high neat strength at 7 days, and little or no increase of strength at 28 days, or even retrogression, is dangerous. Most engineers are inclined to look at such a cement as a doubtful product, and many condemn it. The following is quoted from one author on cement:

"The following rules for acceptance or rejection of material on the results of the tensile test represent conservative practice:

"At seven (7) days: Hold for twenty-eight (28) days on excessively high neat strength.

"At twenty-eight (28) days: Reject on retrogression in neat strength if there is any other indication of poor quality."

The writer will discuss only the question: Is the retrogression an indication of poor quality? Other indications of poor quality will not be considered.

* Presented at the meeting of November 15th, 1911.

First, he will examine the principal ingredients of a Portland cement, and, after analysis, will show the relation of each ingredient to the whole, or finished product.

All investigators agree that the essential ingredients of Portland cement are lime (CaO), silica (SiO_2), and alumina (Al_2O_3). The principal factor is the lime, and the more combined lime present in a cement the greater will be its strength. The lime limit is reached when there is more lime than will combine chemically with the silica and alumina. High lime cements usually set slowly but harden rapidly, and reach their maximum strength in a short time.

When the raw materials are brought together in the kiln, some of the lime combines chemically with the silica, forming calcium silicate, and some of it combines with the alumina, forming calcium aluminate. These chemical combinations must take place in definite proportions, a lack of combination making a more or less unsound product, in accordance with the free lime present. In slaking, lime expands, so that an excess of lime, of more than will unite with the silica and alumina, will cause the cement to expand, or "blow," as it is technically termed, and crack.

Next to the lime, the most important constituent is the silica. The true strength and hardening of a Portland cement is due to calcium silicate. The silica should exist entirely in a combined state, and not as quartz sand, in which condition it does not combine with the lime, and is inert in the subsequent reactions. A Portland cement, therefore, should tend toward a composition approximating the proportion: Lime, 76.6% and silica, 26.4%, as lime and silica combine as tricalcic silicate ($3CaO,SiO_2$).

From the following analysis of an average California Portland cement, it will be seen that cements, as made, do not conform to this. Alumina and ferric oxide are always present in considerable quantities.

Analysis of an
Average California Portland Cement.

Lime (CaO)	62.85
Silica (SiO_2)	22.25
Alumina (Al_2O_3)	7.65
Ferric oxide (Fe_2O_3)	3.35
Magnesia (MgO)	0.78
Sulphur trioxide (SO_3)	1.34

The magnesia in a Portland cement is supposed to replace some of the lime, but it does not normally combine with silica or alumina at the clinkering heat used in the manufacture; and in all probability it is an inert and valueless constituent, and is present in an uncombined state.

Some of the alumina is replaced by ferric oxide, and both are considered as producing the same effect. To the calcium aluminates of a cement are ascribed the initial setting properties. A decrease in alumina tends to make a cement set more slowly, while high alumina affects it in the opposite way. The initial tensile strength of the cement is due to the calcium aluminates, but this strength is probably not permanent.

Several compounds of sulphur are present in cement; the chief of these is calcium sulphate. The action of calcium sulphate on cement is to delay the set. For this reason it is always added (in the form of gypsum, or plaster of Paris) to the cement after burning.

All Portland cements show retrogression in the neat strength at some period, usually between 30 and 90 days.

Examples are shown in Tables 1 and 2, taken from the records of the Laboratory of the Southern Pacific Railroad Company, in San Francisco. Table 1 is compiled from tests of Portland cements manufactured in the United States, and Table 2 from tests of Portland cements manufactured in Europe. The figures represent the tensile strength, in pounds per square inch, and the average obtained from three briquettes broken at the indicated age.

TABLE 1.—American Portlands.

Laboratory number.	Age.						
	7 days.	1 month.	2 months.	3 months.	6 months.	9 months.	12 months.
324	718	817	878	803	870	880	883
325	817	916	816	801	860	879	926
327	780	883	738	771	840	858	867
340	723	753	754	729	729	741	770
395	779	800	750	801	832	855	889
396	825	884	878	741	741	758	775
Averages..	774	842	802	774	812	828	852

The retrogression shown by Tables 1 and 2 is attributed by some authors to the presence of calcium sulphate. Experiments with

cements to which no gypsum had been added, and the same cement with the addition of gypsum, do not indicate this. Sulphates in large quantities will cause a higher short-time test, but the small percentage of sulphur trioxide in cements is not harmful.

TABLE 2.—EUROPEAN PORTLANDS.

Laboratory number.	AGE.						
	7 days.	1 month.	2 months.	3 months.	6 months.	9 months.	12 months.
197	585	667	690	648	645	675	690
240	480	564	728	645	657	671	683
314	672	836	637	654	661	683	696
317	557	663	725	659	667	747	782
354	593	670	655	600	643	653	663
364	581	666	636	612	672	684	706
Averages..	578	678	678	636	658	686	703

This retrogression is probably due to the calcium aluminates. The first strength is obtained from the calcium aluminates, as they hydrate quickly, but the permanent strength is due to the calcium silicates, which are much slower in hydrating.

The full tensile strength of the calcium aluminates is reached at one month, when they become more or less inert; at this age the calcium silicates are acting slowly, but are not strong enough to maintain the strain already reached by the calcium aluminates, hence retrogression takes place.

The calcium silicates are constantly increasing in strength for an indefinite period, and gradually pick up the lost strength. Fig. 1 is a graphic description of this action.

DIAGRAM SHOWING ACTION OF CALCIUM ALUMINATES AND SILICATES.

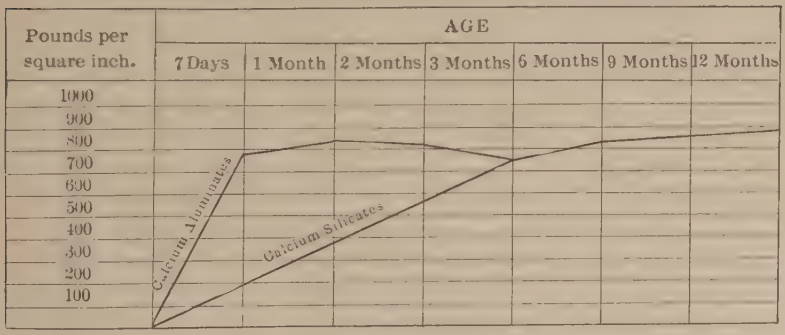


FIG. 1.

If the action of the calcium silicates can be accelerated, so that the final strength is reached sooner, it is an advantage. This is attained, in American Portland cements, by grinding the materials much finer than in former years, and carrying a higher calcium silicate factor. The fine grinding accelerates the hydration of the calcium aluminates and silicates, so that great strength is likely to occur at the 7-day period, and little or no gain, or even retrogression, at the 28-day period.

If the silicates pick up the strength between 1 and 2 months instead of at 6 months, the product is evidently better than the cement that is slow in attaining final strength.

Cements, as manufactured to-day, often show retrogression between 7 and 20 days, and still have an increase in strength between 7 and 28 days.

Generally, there is no retrogression after the 28-day period, which is an indication that the silicates have hydrated rapidly, and permanent strength is reached at 28 days. Table 3 is compiled from tests of cements acting as just stated.

TABLE 3.—Portlands Showing Early Retrogression.

Laboratory number.	Age.						
	7 days.	10 days.	20 days.	1 month.	2 months.	3 months.	6 months.
515	767	809	792	816	838	867	867
517	736	758	875	844	858	865	868
520	748	802	766	783	801	844	857
522	772	736	732	786	800	826	848
542	686	773	660	770	791	840	851

As it is not customary to make the breaks between 7 and 28 days, a cement would undergo retrogression between the periods specified without being detected, and would conform to the specifications.

If this cement is fit for use, why reject one showing retrogression at 28 days?

If retrogression in tensile strength again occurs at a later period, it is only an indication that complete crystallization has taken place, and does not indicate a disrupting action. As crystallization takes place, the cement becomes more and more brittle, and consequently tensile stresses break it more easily.

The fact that the early retrogression in tensile strength is due to calcium aluminates crystallizing quickly, is demonstrated by test pieces in compression, where no retrogression occurs.

DISCUSSION

Mr. Parsons.

MAURICE G. PARSONS, JUN. AM. SOC. C. E. (by letter).—One hesitates to enter into a discussion, with such an experienced cement specialist as the author, as to whether cement should be rejected, preferring to take his decision as final and authoritative. However, the entire subject of "dangerous cement" could well be opened up at this time, as many engineers, perfectly familiar with specified methods of making tests and the required performances thereunder, display lack of ability in interpreting results. It is entirely conceivable that the author's reason for broaching a single phase of a subject fundamentally important to construction was to arouse general interest.

Having had experience in the manufacture and private testing of cement, the writer offers the following comment: Is a cement which shows high neat strength at 7 days and little or no increase of strength at 28 days, or even retrogression, dangerous?

Apparently with the idea of obtaining a full discussion, as more beneficial to the Profession than mere off-hand assertion, the author has gone over the field of neat retrogression without drawing any conclusions, only bringing out the essential points bearing directly thereon. At the same time he gives free latitude to others, with the result that rejection may be considered in a broad sense.

The following considerations would influence a rational discussion:

1. Although the experimental stage in the manufacture of Portland cement—the time when "improved naturals" were being attempted—is now past, still a poor batch will creep through the kilns in spite of the utmost vigilance on the part of the mill laboratory; an excess of lime (due largely to the manufacturer's effort to obtain a product of high strength) being a frequent cause of trouble. This, or any other grave fault, will manifest itself in other tests than that of neat strength. The neat strength test is severe.

2. Even careful operators sometimes place briquettes in the jaws so as to obtain really an unfair break on a specimen brittled to some extent by 28 days. Furthermore, while the test is in progress, the cement is continually aging and thus perhaps losing its poor quality.

3. The strength is directly dependent on the fineness, especially the early strength, which is so desirable and is such a good point in Portlands. Moreover, brittleness and ultimate strength are reached sooner by a finely ground cement.

4. Next to the boiling test, sand strength and not neat strength is in the end the true criterion. Pure cement is not used in practice, and, frequently, the sand strength will continue to increase even though the neat strength has retrograded.

In view of these points, together with the author's remarks (special emphasis being placed on the fact that cements may retrograde

Mr. Parsons. and pick up again before 28 days, which indicates the possibility that any perfectly safe material is apt to retrograde some time early in its life), the writer certainly would not condemn a cement because of a small neat retrogression at 28 days, unless there are other bad indications or the concrete is to be used where great dependence is placed on it. Even then, an expensive delay would weigh heavily on the side of acceptance. Conditions permitting a re-test should be provided, as this would be fairer than absolute rejection.

Mr. Richardson. CLIFFORD RICHARDSON, M. AM. SOC. C. E. (by letter).—This subject has been discussed at great length in the past. There are many engineers who believe that a cement which shows retrogression in the tensile strength of neat briquettes is undesirable, but there are probably more who do not agree with such a statement. The writer believes that little attention should be paid to such an occurrence if the cement is sound in every other way, and has an adequate strength at 28 days. Cements which show a retrogression are usually high in alumina, and although they may have such a characteristic when tested neat, this will not be the case with mortar, or if the crushing strength is made the criterion of the value of the material. This has gone so far in Germany that the tensile strength has been abandoned as a means of determining the value of cement, and crushing tests are the standard. Tests of cement in the neat form, without the addition of an aggregate, are of no great value, as cement is never used in this form in construction.

Mr. O'Hara states that a Portland cement should tend toward a composition approximating the proportion of lime and silica in tricalcic silicate. This can hardly be considered a correct assumption, as such a silicate would have little strength or value as a cementing material. It is the presence of aluminate which gives Portland cement its value, although, if the latter is present in too large a quantity, the cement may be undesirable.

The cementation index of the California Portland cement (an analysis of which is given by Mr. O'Hara), as calculated by the formula of Eckels, is 0.99. It is usually considered that when this index falls below 1.0 the cement must necessarily contain free lime or free magnesia, but, as in this case, it is so near unity that the cement can be regarded as normal.

Mr. O'Hara's data in regard to cement tests in the laboratory of the Southern Pacific Railroad Company, in San Francisco, are unfortunately confined to results of breaking briquettes of neat cement. If they had been extended to tests of 3:1 mortar, and to the crushing strength of the cement, the data would be of greater value for determining the quality of the cement. As it is, they do not afford a means of determining whether or not the cements in question are undesirable.

In the writer's opinion, there is no reason to condemn them because they show such retrogression in a neat form, as in Tables 1 and 2. Mr. Richardson.

R. S. Greenman, Assoc. M. Am. Soc. C. E.—Mr. Richardson's statements, cautioning against laying too much stress on tests of neat cement, are very interesting. It is easy to agree with him in that particular. Believing that much reliance should not be placed on neat tests, several series of tests with sand were made, some five or six years ago, to cover a long time, with the idea of ascertaining whether retrogression would take place with sand briquettes in the same way as with neat briquettes. Five of the best known or "standard" brands were used, and the number of briquettes made was sufficient to give an average of ten for each test. It is understood that Mr. O'Hara had Mr. Greenman.

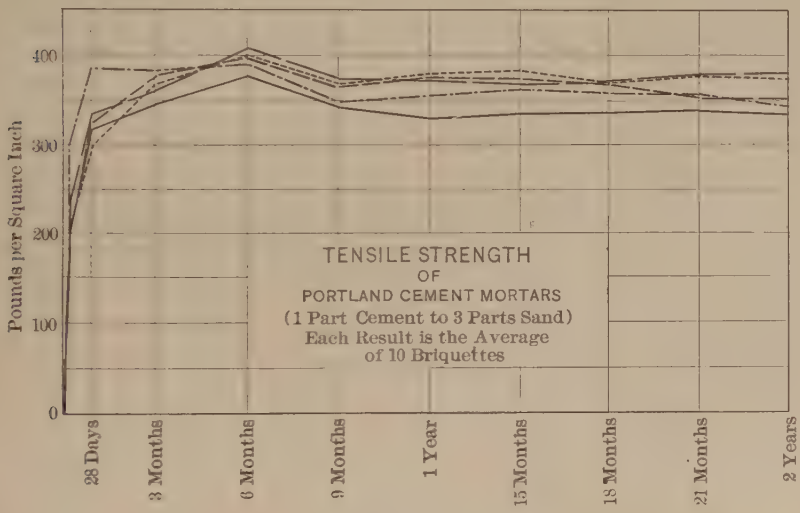


Fig. 2.

only three briquettes for each test. The personal equation in the testing of cement is so variable that three briquettes are hardly enough, therefore ten were taken for each test, and enough were made to last for two years. Contrary to opinions expressed in the paper, and by some who have taken part in the discussion, it was found that there was retrogression in the sand briquettes between the periods of 6 months and 1 year—as a rule, at about 9 months. From that point up to 2 years the curve was wavering.

To confirm these tests, others were made by another operator, using another set of cements, and, as was expected, they acted in the same way, showing that retrogression is common, not only in neat tests, but in sand tests.

Mr. Greenman. The speaker is not quite ready to agree with Mr. O'Hara in his explanation of the reason for retrogression. Because of the new idea regarding cement as a "colloid," some of the opinions in regard to the form of crystallization of cement are necessarily open to doubt. Without trying to explain the retrogression, the speaker would merely endorse the expressed opinion that, in determining the quality of the cement, too much stress should not be laid on the retrogression in tests of neat cement. Noticeable retrogression in short-time tests, however, should be regarded as serious.

Mr. Goodrich. E. P. GOODRICH, M. AM. SOC. C. E.—Of course, the proportion of flour in the cement has a very great effect on the time of setting. The fineness tests, as ordinarily given, do not include that for extreme fineness, or the quantity of flour contained in the cement. The speaker has tested cement by blowing the finer materials out, and it is a well-known fact that these very fine particles set practically instantly. Perhaps this particular cement, which set so quickly, contained a large quantity of these fine materials; that fact would not be discovered by any of the chemists, or by any of the physical tests likely to be made in the laboratory.

A very interesting series of experiments has been carried on in the geo-physical laboratory by Mr. Shepherd, in which the physical composition of Portland cement has been practically determined. It is not quite as Mr. O'Hara describes it, the composition of various Portland cements probably crossing two or three lines of chemical demarcation, so that it is very improbable that any two cements will be identical in chemical composition. The quantity of lime has considerable effect on the strength and on the setting qualities. The speaker has made tests of cements in which there was a long, slow increase of strength from 7 days up to 6 months—a parabolic line—and he has seen tests of other cements in which the 7-day test was practically the maximum, and the strength was indicated by a horizontal line running out indefinitely, with no retrogression. The average of several tests of most cements will show no retrogression, while certain samples will show marked retrogression, coming up again at the end of several months.

There is great doubt in the speaker's mind as to whether or not the aluminate suggestion made by Mr. O'Hara is a correct one. Spackman has made many tests on cements consisting almost entirely of calcium aluminate, which did not show such retrogression. This idea of the combination of the calcium aluminate and calcium silicate, the speaker believes, is due to either of two well-known cement experts, or perhaps to both, conjointly, in a certain sense, and they put it forward simply as a possibility, not at all as an actuality, or even as something which they had shown by their own tests; but Mr. O'Hara

puts it forward as his explanation of the matter. One possible cause is free lime. Mr. Goodrich.

The speaker knows of one case of a cement in a very large structure, in which the cement tested properly at the mill, but had a flash set on the work; one week later it was testing properly; one month later it would flash set again, and eventually it came out all right. This was traced to the proportion of free lime and the later slaking of the lime, with the slower effect of the gypsum which is always added. S. B. Newberry, Assoc. M. Am. Soc. C. E., in one of his papers before the Manufacturers Association, has explained the matter very satisfactorily.

E. D. Knap, Assoc. M. Am. Soc. C. E.—In some laboratory tests which came to the speaker's attention about 10 years ago, a certain quick-setting cement had to be mixed with ice water, and, even then, required rapid handling in order to get it into the moulds before it set. This cement would have been rejected at once, but, as it was badly needed on the work, it was decided that a batch of mortar should be made with it and tested in the construction of a brick arch. Four bags of it were made into mortar, with the proper proportion of sand, the mixture being quite wet; then, until it was used, a man kept turning it over and adding water from time to time as required. It was used in a little less than an hour, and its action in the brickwork was entirely normal, though it set so rapidly in the laboratory that it was thought to be valueless. The entire shipment of this cement was accepted, and the arches built with it are still standing and doing the work for which they were designed. Mr. Knap.

In the following cases the speaker will confine himself entirely to the results of neat cement tests, because Mr. O'Hara deals with them exclusively. There is more variation in these results than one would think possible. For instance, of one hundred lots of a certain cement, the maximum strength in five cases was attained at 7 days; in thirty-six cases it was at 28 days; in thirty-eight cases it was at 3 months; in twelve cases it was at 6 months; and in nine cases it was at 1 year. This was considered a first-class cement, and many thousand barrels of it were used in the work for which these tests were made. The curves indicating the results of the tests, from 7 days to 1 year, had several peaks and hollows. The tests at 1 year in most cases gave higher results than those at 9 months, but the strength at 1 year was not the maximum, except in the few cases noted—nine in one hundred.

The results of tests on one hundred lots of a second brand show that in eighteen cases the strength at 7 days was greater than at 28 days. Mr. O'Hara states that there is seldom any retrogression after 28 days, but, of this brand, 42% was stronger at that time than at

Mr. Knap.

subsequent tests, and yet it was probably a normal cement, and perfectly safe when properly used.

Of thirteen lots, of a third brand, only two had a greater strength at 7 than at 28 days; in three cases the maximum strength was at 28 days; in four it was at 3 months; in two it was at 9 months; and in one it was at 1 year. In six of these cases the strength at 1 year was less than at 9 months, and in four it was greater; in the other case the series ended with the 9-month test, which was also nearly the maximum. In one case this brand was tested at 2 years and showed an increase in the year of about 140 in 800 lb.—quite a large percentage.

Of a fourth brand there were eleven lots, and nine of them were stronger at 2 years than at 1, and yet the results between 7 days and 1 year were quite variable.

Neat tests in the laboratory do not give a fair indication of the value of a cement, because the conditions are so different from those which obtain in the field, where the quality of the sand, the quantity and perhaps the quality of the water, the method and speed of handling, and many other factors, tend to produce results which differ materially from those of the laboratory. The speaker has known of cases where cement failed to pass the steam test—one of the most reliable methods of indicating danger—and specimens disintegrated in the air in 7 months, and yet that cement was used, and to-day, as far as can be seen, is perfectly good.

The cement specifications should lay greater stress on fine grinding, by stipulating that a larger percentage shall pass the No. 200 sieve, as that would improve the cementing value of the product when used with sand.

In all mortar tests, the sand used should be that with which the cement is to be mixed on the work; in this way both the sand and the cement would be tested. Of course, this is somewhat at variance with accepted ideas in regard to cement testing, but it is very desirable in many cases to know the actual results in the field, rather than those secured under laboratory conditions. For example, in compression tests of concrete the speaker has obtained quite different results with cubes made from the concrete in the field and with those made in the laboratory with the same relative proportions of clean stone and sand of the best quality. One set represented working conditions; the other represented ideal, theoretical, laboratory conditions. These cube tests proved very conclusively that the aggregates used by the contractor on that work were not as good as could have been desired.

The speaker is of the opinion that Mr. O'Hara has not sufficient information on which to base his conclusions regarding the effect of chemical composition on the setting qualities of cements. Deductions

should be based on the results of tests of not less than several hundred lots of cement, of various compositions. If "a little learning is a dangerous thing," surely deductions from insufficient data are far more dangerous, because, in the absence of evidence to the contrary, they may carry more weight than the data warrant, even if they are not entirely misleading. Mr. Knap.

In urging that radical changes be made in the specifications for cement testing, the following suggestions are offered:

1.—That all neat cement tests for tension and compression be abandoned;

2.—That a series of 1:3 mortar briquettes be made (using crushed quartz or Ottawa sand) in order to connect the results with former records, and with compression tests on a series of small cylindrical specimens having a diameter equal to half their height.

For all mortar tests these cylinders should be made with the same sand as that to be used on the work, and with as nearly as possible the same proportions of sand and water. For tests of concrete, larger cylinders should be made, using the same sand and stone as that to be used on the work, and also in the same proportions, and, as nearly as possible, of the same consistency or wetness. The tests for mortar and concrete should cover the same periods as those now specified. This need not involve much, if any, more labor than for the present requirements, for one compression specimen per period will give more reliable results than the average of quite a number of tensile tests; and, after a time—when the relation between the crushed quartz or Ottawa sand briquettes and the mortar or concrete cylinders has been thoroughly established for the various cements—the tensile tests can be discontinued.

J. M. O'Hara, Assoc. M. Am. Soc. C. E. (by letter).—The title of this paper should have been "Retrogression in the Tensile Strength of Neat Cement." Mr. O'Hara.

The commercial laboratories on the Pacific Coast condemn cement if it fails to pass the Am. Soc. C. E. Specifications in any particular. The writer selected neat tests because such tests are used as a basis for the rejection of many shipments of good cement. The specifications should be modified regarding neat tensile tests.

Mr. Parsons expressed the writer's object in presenting the paper when he said:

"Apparently with the idea of obtaining a full discussion, as more beneficial to the Profession than mere off-hand assertion, the author has gone over the field of neat retrogression without drawing any conclusions, only bringing out the essential points bearing directly

Mr. O'Hara.

thereon. At the same time he gives free latitude to others, with the result that rejection may be considered in a broad sense."

Tensile tests of neat cement are not essential in routine work, as the results are not readily interpreted, except by experienced testers, but they are certainly necessary in research work. Sand tests were not presented, as the sole object of the paper was to discuss neat cement.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1215

EXPERIMENTS ON THE FLOW OF WATER IN WOOD STAVE PIPES.*

By E. A. Moritz, Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. Robert G. Dieck, Melvin L. Enger, J. L. Campbell, Rudolph Hering, Gardner S. Williams, J. S. Moore, Charles G. Darrach, and E. A. Moritz.

Introduction.—This paper is a description and discussion of certain experiments on loss of head in wood stave pipes, made under authority of the United States Reclamation Service, on the Sunnyside Project, in the State of Washington. The experiments had their inception in the following circumstance, which shows that necessity is the mother of investigation, as well as of invention:

In the spring of 1908 the writer was called upon to determine the required size of a wood stave pipe to carry water across the Yakima River to irrigate some 9 000 acres of land under the Sunnyside Project. Having had no previous experience in similar calculations, nor any knowledge of actual flow under similar conditions, he was forced to resort to the published data on the subject. These data were rather meager, and consisted of the following, as listed† by Theron A. Noble, M. Am. Soc. C. E.:

"1.—By James D. Schuyler, M. Am. Soc. C. E., on a 30-in. pipe line at Denver, Colo., the details of which are not available. The value of n, in Kutter's formula, is given as 0.0096.

"2.—By Mr. Frederick P. Gutelius, on the 24-in. pipe of the Butte City Water Company, in which he confirms the use of 0.01 as the value of n, in Kutter's formula.

* Presented at the meeting of December 6th, 1911.

† "Flow of Water in Wood Stave Pipes," *Transactions*, Am. Soc. C. E., Vol. XLIX, p. 134.

"3.—By Arthur L. Adams, M. Am. Soc. C. E., on an 18-in. pipe for the City of Astoria, Ore. The details of this experiment are given. The results show a value of $n = 0.01$, and $c = 132.9$. Only one observation was taken at a velocity of 3.605 ft. per second.

"4.—By Arthur L. Adams, M. Am. Soc. C. E., on a 14-in. pipe for the West Los Angeles Water Company. In all, there were seven observations, the velocity varying from 0.691 to 1.531 ft. per second; value of c, from 99 to 113; value of n, about 0.011.

"5.—By Messrs. Marx, Wing and Hoskins, on the 72-in. pipe of the Pioneer Power Plant, at Ogden, Utah. This is by far the most complete and satisfactory set of experiments yet conducted to determine the loss of head in wood pipe. The results have been fully written about and discussed.

"6.—By the writer, on 44-in. pipe, as described in this paper.

"7.—By the writer, on 54-in. pipe, as described in this paper."

These tests are replete with inconsistencies, as brought out in the various discussions on them; and any attempt to co-ordinate the results is apt to be discouraging. On the whole, no well-defined fault was found with any of the published experiments, and this makes an attempt at successful analysis and co-ordination all the more difficult. However, in the above instance, the writer interpreted the results as best he could, and calculated the size of the pipe required for the case in hand.

On the Sunnyside Project of the United States Reclamation Service there are quite a large number of wood stave pipes in service, and the above incident gave the writer the idea that experiments on friction losses in these pipes could be made with very little expense, in connection with the regular hydrographic work on the project, and bade fair to produce valuable information. Accordingly, the idea was suggested to E. McCulloh, M. Am. Soc. C. E., Project Engineer, and met with his instant approval; and it is largely due to his hearty co-operation and unfailing interest that a considerable amount of valuable information has been gathered. The experiments and the reduction of the observations were in direct charge of the writer, who was ably assisted by Mr. J. S. Moore, Junior Engineer, Hydrographer on the Sunnyside Project, to whom a large measure of credit is due, not only for good work performed, but for his many valuable ideas and suggestions in connection with taking and reducing the observations.

The irrigation season on the Sunnyside Project begins on April 1st and ends on October 31st, and the experiments herein described were made during the seasons of 1909 and 1910, principally in the last two months of each season, because at this time a greater variation can be made in the flow of the various pipes without interfering seriously with the process of irrigating. Even at this time it was not possible to vary the flow as much as was desired, on account of the lateral at the intake or outlet not being able to take care of extreme variations in the water supply. However, this difficulty probably had no effect in vitiating the results of the experiments, for it is not likely that the conclusions would have been different had greater variations been possible. As it was, the field of velocities, from 0.4 to 6 ft. per sec., was quite thoroughly explored. All statements made herein must be considered as applying to velocities within these limits.

Scope of Tests.—Observations were taken on pipes of the sizes and lengths given in Table 1.

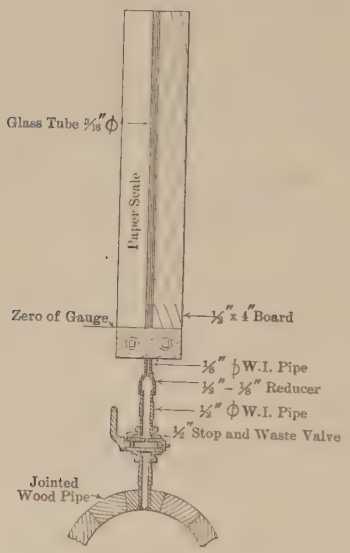
TABLE 1.—Sizes and Lengths of Pipe.

Year.	Diameter, in inches.	Length, in feet.	Year.	Diameter, in inches.	Length, in feet.
1909	55¾	2 848.2	1910	55¾	2 848.2
"	14	3 637.0	"	22	2 087.0
"	8	3 394.5 and 2 002.0	"	18	2 802.5 and 2 774.2
"	6	500.4	"	14	3 637.0
"	5	1 822.0	"	12	2 022.0
			"	8	3 515.2 and 4 054.9
			"	6	1 801.0 and 1 892.0 and 2 021.0
			"	4	1 109.5 and 603.0

The 55¾-in. pipe is of continuous-stave construction, and the others are of the so-called machine-banded, jointed-stave type, manufactured in usual lengths of from 8 to 20 ft., with mortise and tenon ends, which are driven together in the trench and not otherwise jointed.

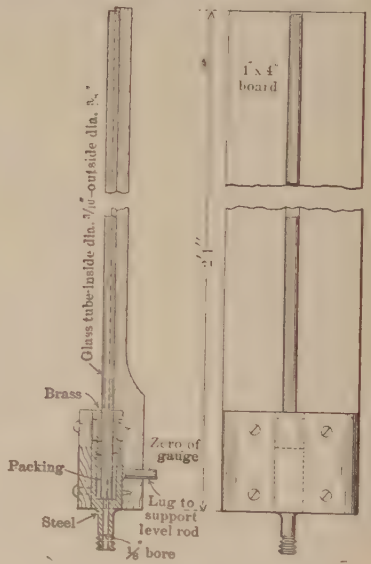
The range of velocities was between a minimum of 0.403 ft. per sec. for the 6-in. pipe tested in 1910, and a maximum of 5.874 ft. per sec. for the 8-in. pipe tested in 1910. As has been stated, this

range was limited in some cases by the required regulation of irrigation water, but in some cases it was also affected by such items as the difficulty of measuring larger quantities, the presence of air in the pipe, etc. In the 5- and 6-in. pipe this latter difficulty was especially troublesome, and on this account, and because of its short length, it was thought best not to test this 6-in. pipe again, therefore the 1910



WATER GAUGE ATTACHMENT TO WOOD PIPE

FIG. 1.



MANOMETER SCALE

FIG. 2.

experiments were made on a different and longer one. No other 5-in. pipe was available, but the tests on the pipe of that size, made in 1909, were not repeated. Observations made in 1909 were repeated in 1910 on the 55¾-, 14-, and 8-in. pipes, the identical lengths being used.

APPARATUS AND MEASUREMENTS.

Pressure Measurements.—The pressures were measured with water columns for the lower heads and mercury columns for the higher heads, when scaffolding would have been necessary in order to use a water column.

Fig. 1 is a diagram of the water gauge attachment, and requires no explanation. The paper scales on which the heights were observed were graduated to 0.01 ft., and readings were recorded to 0.001 ft.

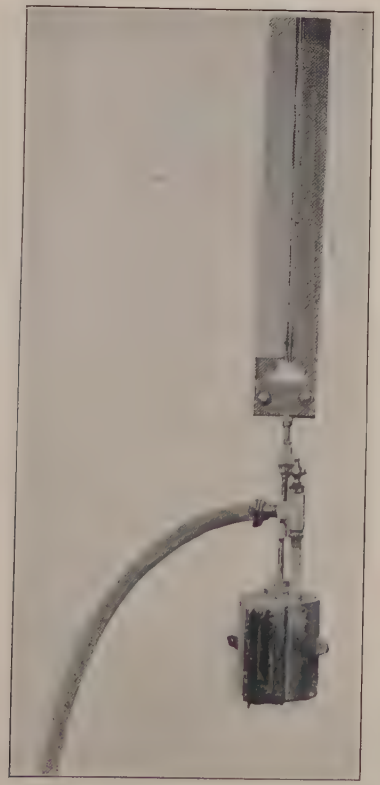


FIG. 1.—SMALL MERCURY RESERVOIR AND GAUGE.

FIG. 2.—LARGE MERCURY RESERVOIR AND GAUGE.

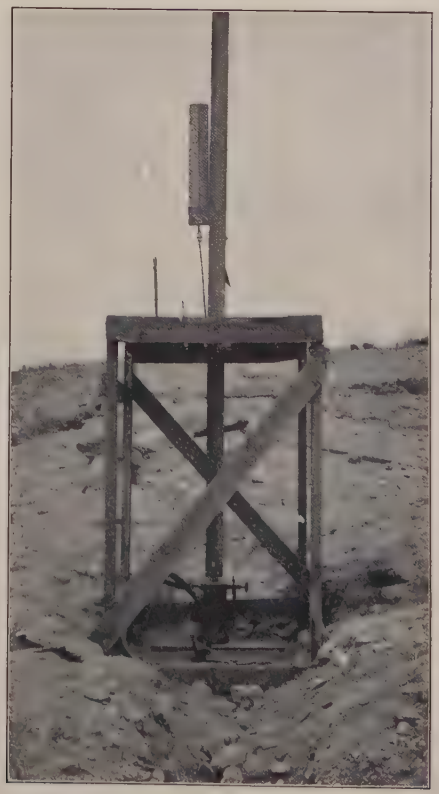


FIG. 3.—MANOMETER NO. 4, ON 55¾-INCH PIPE.

The scale shown in Fig. 1 was used in 1909. This was improved somewhat for the 1910 experiments, as shown in Fig. 2. Setting a level rod on the zero of the gauge, as used at first, was found to be awkward, and, on the later instrument, a lug at the back was provided for this purpose. The graduations were also marked directly on the wood, instead of on a paper glued to the wood. Figs. 1 and 2, Plate LVI, show the mercury reservoirs of the two types used. These are the same gauges as Messrs. Marx, Wing, and Hoskins used in their 72-in. pipe experiments,* and were obtained for the tests of these Washington pipes through the courtesy of C. D. Marx, M. Am. Soc. C. E. They were set just as shown on Plate LVI, except that, when required for higher pressures, an additional length of ⅛-in. pipe was inserted between the reservoir and the glass gauge. Fig. 3, Plate LVI, shows one of the pressure stations on the 55¾-in. pipe. The larger reservoirs, shown in Fig. 2, Plate LVI, were used on the 55¾-in. pipe; and, on all the smaller sizes, when a mercury column was necessary, the smaller reservoirs were used. The specific gravity of the commercial mercury in the gauges, was determined by a set of tests made in the physical laboratory of the High School at Sunnyside, Wash., and was found to be 13.58 at 15° cent.; this value was used in all the reductions.

Attachment of Gauges.—The gauges were attached to the pipes by a ½-in. wrought-iron nipple, threaded at both ends and screwed into a hole bored in the wood pipe, in the manner described by Mr. T. A. Noble,† that is, the hole which was to receive the nipple was not bored of full size clear through the pipe, but was stopped as soon as the tip of the bit projected through the inner surface. It is believed that a hole made in this manner is not as likely to have rough edges and projections on the interior surface—which might cause the velocity head to affect the pressure measurements—as a hole of full size bored clear through. The hole at Manometer No. 3, on the 55¾-in. pipe, bored in this manner, was observed from the interior of the pipe and found to be perfectly smooth. The hole at Manometer No. 4 could not be detected from the inside, which makes it reasonable to suppose that the smoothness of the interior surface at the tap was all that

* Described in detail in *Transactions*, Am. Soc. C. E., Vols. XL and XLIV.
† "Flow of Water in Wood Stave Pipes." *Transactions*, Am. Soc. C. E., Vol. XLIX.

could be desired. On account of the small size of all the other pipes, it was impossible to observe the taps from the interior.

All connections were through a single tap at the top of the pipe. For observations at the higher velocities, on the 18-in. pipe, the disturbances due to the pressure of air were so violent that it was found necessary to make the connection at a lower point. This is the only case where the point of connection was varied.

Discharge Measurements.—In all the experiments the quantities were measured over weirs. Those for the 55¾-in. pipe were measured over an 18-ft. rectangular weir with a crest of No. 11 gauge iron. This weir was at the intake of the pipe. Plate LVII is a view of the weir and

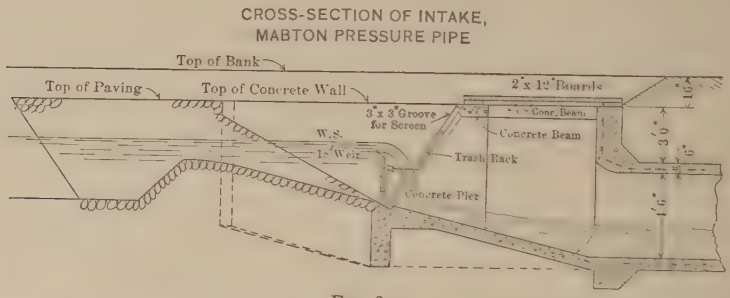
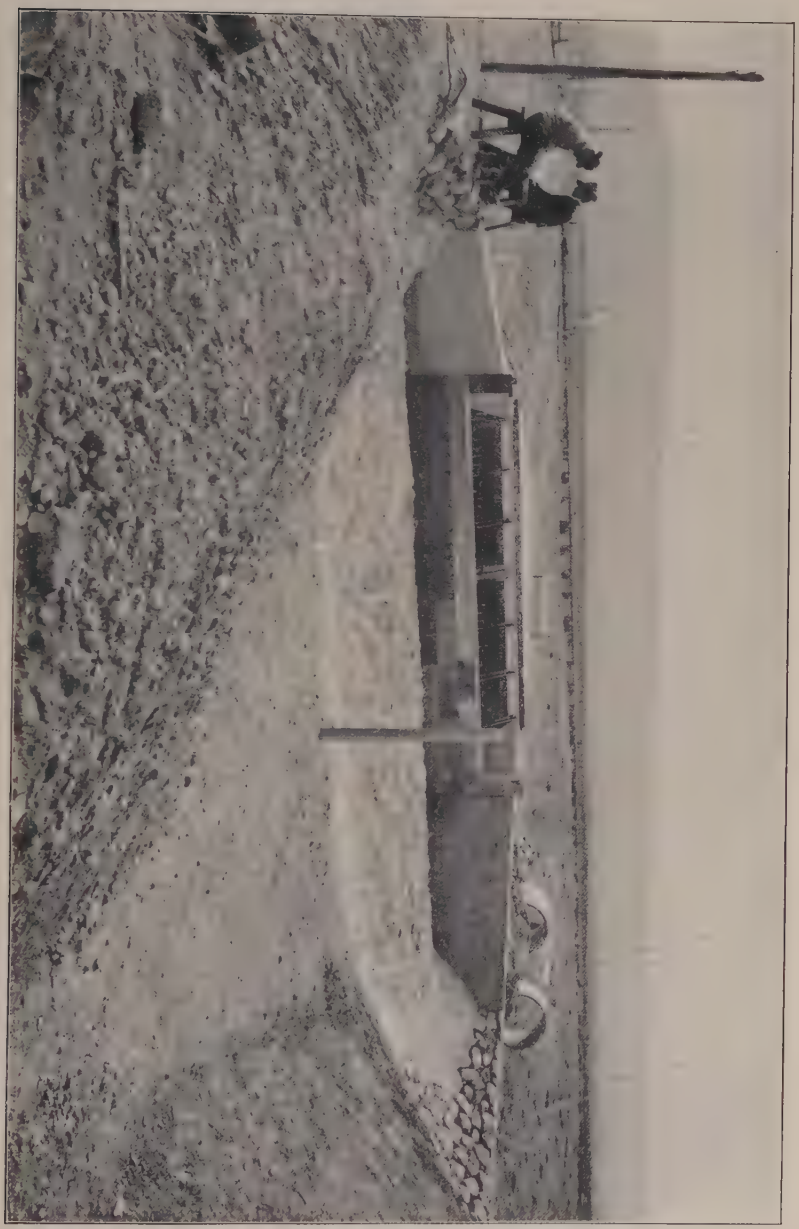


FIG. 3.

intake, and Fig. 3 is a cross-section. During the 1909 observations there was considerable velocity of approach above the weir. This was estimated by taking eight-tenths of the surface velocity, measured by floats, as the average velocity. In order to determine this velocity of approach more accurately, and with the hope of obtaining a check on the weir formula, measurements of discharge were made for the 1910 observations with a Price current meter. The meter was operated from a bridge across the canal about 75 ft. up stream from the weir. For heads higher than 0.8 ft. over the weir, Smith's formula for contracted weirs agreed very closely with the meter results, but, for all lower heads, no formula seemed to give quantities as high as the current meter. The channel in front of the weir was badly silted up during all the observations, and it was difficult to determine the velocity of approach, especially for the smaller quantities. It has been concluded, therefore, that the current meter measurements are the more reliable, especially in view of the fact that uniformly good results had been



INTAKE OF 55¾-INCH PIPE.

obtained previously with the current meter in other channels. The quantities determined by the current meter have been used in the reductions. The discharge curve obtained from the meter results is plotted in Fig. 4, with the observed heads over the weir as ordinates. The curve obtained from the weir measurements, by using Smith's coefficients for contracted weirs and correcting for velocity of approach, is shown by the dashed line in the same figure.

The discharges from all other pipes were measured over Cippoletti weirs. No attempt was made to measure velocities of approach, but care was taken to erect a suitable baffle-wall across the approach

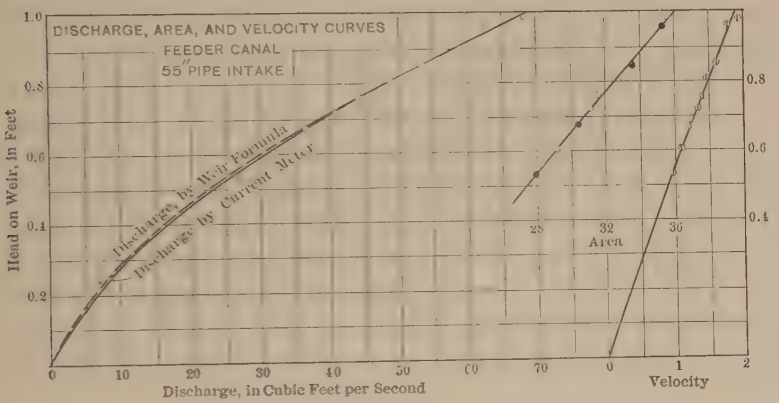


FIG. 4.

channel in all cases when the velocity of approach was likely to assume dangerous proportions. Fig. 5 shows a cross-section of the sharp-crested weir most frequently used; in a few cases, however, it was necessary to take the measurements with the weir already in place, a typical cross-section of which is also shown in Fig. 5. The latter was the standard on the Sunnyside Project until about two years ago, and a large number are still in place. For the quantities measured over these weirs, reductions were made on the basis of a series of volumetric tests made on the Sunnyside Project by John C. Stevens, Assoc. M. Am. Soc. C. E.* For Observations 1 and 2 on the 22-in. pipe, it was necessary to use an 8-ft. weir of this same construction, and for these an addition of 2% to the discharge was considered to be a reasonable

* *Engineering News*, Vol. 64, p. 171.

value. For the 6-in. pipe tested in 1909, a similar weir with a crest length of 1 ft. was used, but the quantities were derived from a comparison of discharge previously measured during the observations on the 5-in. pipe between a 1-ft. weir of this type at the intake and a sharp-crested one at the outlet. On account of the abnormal condition of these crests, the result obtained from this comparison is probably nearer the true value of the discharge than that determined by a comparison with the volumetric measurements previously mentioned. With the exceptions just mentioned, all pipes were equipped with sharp-crested weirs, reduction to discharge being made directly from

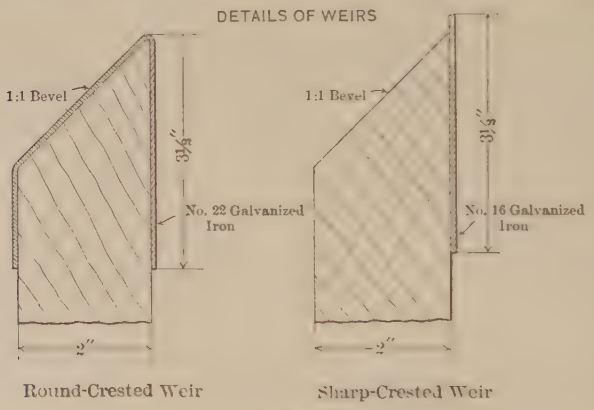


FIG. 5.

the Cippoletti formula, $Q = 3.37\ L\ H^{\frac{3}{2}}$. In all cases, except for the tests made in 1909 on the 14-in. pipe, the heads were measured with a hook-gauge, on which the readings were taken to the nearest thousandth of a foot.

Elevation and Distance.—The difference in elevation between zeros of gauges was determined with a wye-level. A closed circuit was run in every case, and when apparent discrepancies or inconsistencies were encountered, which might have been attributed to errors in elevations, the circuit was repeated, but in no cases were the levels found to be in error. The limit placed on the error of closure was

$$0.01\ \sqrt{\text{Distance, in Miles,}}$$

and the average error of closure for all pipes experimented on was

$$0.005\ \sqrt{\text{Distance, in Miles.}}$$

Distances were measured with a 100-ft. steel tape.

Observations.

Plan.—During a test observers were stationed at each manometer and at the intake. Most of the pipes are in open and gradually sloping country, and each observer was able to see the others, either with the naked eye or with a field-glass, so that it was possible to signal back and forth at will. After all the measuring apparatus had been properly attached and tested, and the flow into the pipe approximately adjusted to a desired quantity, some time was allowed—usually about ½ hour—for the flow to become steady, as indicated by the action of the gauges. Sometimes, due to various causes, a much longer time than this was required before a consistent set of readings could be obtained, and especial care was always taken to make sure that the flow was not under the influence of abnormal conditions. Each test lasted 30 min., during which time the pressure gauges were read at intervals of 1½ min. The gauges nearly always fluctuated slightly, and the maximum, minimum, and average readings were recorded. For each test, therefore, sixty gauge readings were recorded for each manometer. The valve between the pipe and the gauge was always closed down until the fluctuations of the column were almost entirely destroyed, but not enough to involve any danger of shutting off the connection entirely. Thus the fluctuation of the column was reduced to such an extent that the greatest normal vibration was about 0.003 ft., except in the 55¾-in. pipe, where the fluctuations at times were somewhat higher, but never more than 0.006 ft. Greater fluctuations were observed in several cases, but were caused by the presence of air, which was a source of some annoyance in the smaller pipes. In using a water column connected to the top of the pipe, it was practically impossible to read the instruments when any appreciable quantity of air was being carried. When such conditions arose, the test was stopped and steps were taken to prevent air from entering the pipe, no further readings being taken until this source of disturbance was overcome.

Reductions of Observations.—As has been stated, the discharges of the 55¾-in. pipe were measured with a current meter in an earth channel. The reduction of these observations requires no further comment. The manner of reducing the discharges of the other pipes, and the various corrections applied, have also been explained. A brief

description of the manner of reducing the gauge readings may be of value in interpreting the tabulations:

On account of the comparatively large loss of head in the smaller pipes, it was found to be an unnecessary refinement to apply temperature corrections to the gauge readings; consequently, their reduction to pressure at the station was a simple matter. For the water gauges, the zero of the glass scale was taken as a convenient datum, and the pressure above this point was then equal to the reading of the gauge. For the mercury gauges, the zero of the lower scale on the reservoir was taken as the datum, and the water pressure was then obtained by subtracting the reading of the lower scale from the height of the upper mercury surface above the assumed datum, multiplying this by 13.58, and adding to this product the reading of the lower scale. The pressure, in feet of water, thus obtained, added to the elevation of the assumed datum, gave the elevation of the hydraulic gradient at the station. Having obtained this quantity at two stations on the pipe, the loss of head between them was obviously the difference between these two elevations.

For the 55¾-in. pipe, the total loss of head between the two observation stations was so small, and the difference in length of the two mercury columns was so great, that temperature corrections were desirable.

When the experiments on the 55¾-in. pipe were begun the temperature of the water was about 15° cent., and this was chosen as the standard for the two sets of experiments on this pipe. The reduction to this standard temperature was made by multiplying by the factor, $1 - 0.00018\ (T - 15)$, T being the temperature of the mercury column, in degrees, centigrade, and 0.00018 being the average value of the coefficient of expansion of mercury for all temperatures found in the experiments.

As all the mercury columns were standardized to a constant temperature of 15° cent., any departure in the water temperature from the standard also had to be taken into account. This reduction in the mercury column was made by applying the factor, $1 + 0.00015$ (t — 15), t being the temperature of the water in the pipe, in centigrade, and 0.00015 being the value of the coefficient of water in the neighborhood of 15° cent. The correction

Distances we

(t

degrees,

for the mercury temperature is negative, while that for the water temperature is positive.

The method of applying the temperature correction may be explained by reference to a single observation, as No. 4, of the 1910 series in Table 2: The actual length of the mercury column at Manometer No. 4 was 7.418; mercury temperature, 13° cent.; water temperature, 11° cent. To reduce to an equivalent column at 15° cent., there must be added $7.418 \times 0.00018 \times 2 = 0.0027$ ft. The correction for water temperature was $7.418 \times 0.00015 \times 4 = 0.0044$ ft. Hence the reduced height of the mercury column was $7.418 + 0.0027 - 0.0044 = 7.4163$.

Description of Pipes, and Results of Observations.

55¾-In. Pipe.—This is a continuous wood stave pipe, built by the Reclamation Service in 1908. Water was first turned into it for operation in May, 1909, and at the time the first test was made it had

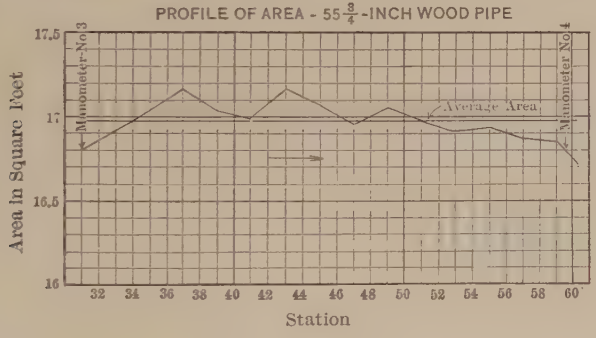


Fig. 6.

been in continuous use for about 5 months. Before the water was turned in, measurements of the diameters were made between Manometers Nos. 3 and 4 (the reach in which losses were determined) at intervals of 200 ft. From these measurements the average area was found to be 16.98 sq. ft., which figure was used in reducing the observations. Four diameters were measured at each point: vertically, horizontally, and at angles of 45 degrees. The shortest diameter measured was $54\frac{1}{16}$ in., in the vertical plane; the longest diameter was $57\frac{3}{8}$ in., in the horizontal plane. Fig. 6 is a profile of the areas. The measurements were made before the pipe was put into active service, and before the wood had had a chance to obtain its ultimate swelling.

To avoid the possibility of any errors in the reductions, due to a change in area after these measurements were made, they were repeated at the completion of the experiments. Corresponding diameters were measured at the same stations, and were found to be equal to the first measurements to the nearest $\frac{1}{16}$ in. This is very surprising, as the pipe had been in service for two years.

Figs. 7 and 8 show, respectively, the alignment and profile of the pipe. The manometers on which observations were made in these experiments are Nos. 3 and 4. The figures show clearly that the conditions were very favorable for good results in this reach. There is no horizontal curve or obstruction of any kind between the two stations, and the vertical curves are light. It will also be noted, from Fig. 6, that the areas at the two manometers were almost exactly equal, thus precluding any error due to inequality of areas at the manometer stations. The various exhibits connected with the experiments on this pipe are shown in Fig. 3, Plate LVI, Plate LVII, Figs. 3, 4, 6, 7, 8, and 12, and Table 2. The table is self-explanatory, and presents a fairly complete account of the observations and the results deduced therefrom. The value of Kutter's coefficient, n, has been calculated for each experiment, and is given in the last column. The relation between the velocity in the pipe and the calculated value of n is shown in Fig. 12.

The condition of the $55\frac{3}{4}$-in. pipe at the time of the experiments may be partly inferred from the statements in reference to the cross-sectional areas. The pipe was new, well rounded out during construction, and contained only minor distortions. There was no deposit of any kind in the bottom, and no growth of any kind on the interior surface; in fact, the interior seemed to be in as good condition after the experiments as the day it was built. These points were determined by a thorough inspection of every foot of the interior between Manometers Nos. 3 and 4, both before and after the experiments were made.

22-In. Jointed Wood Stave Pipe.—This pipe was built in 1906 by the Washington Irrigation Company, and has been in use for four irrigation seasons of 7 months each. During the other 5 months of each season no water was run through it, and it was probably almost, if not entirely, empty during most of this period, as it had several small leaks. The normal quantity discharged during the period of

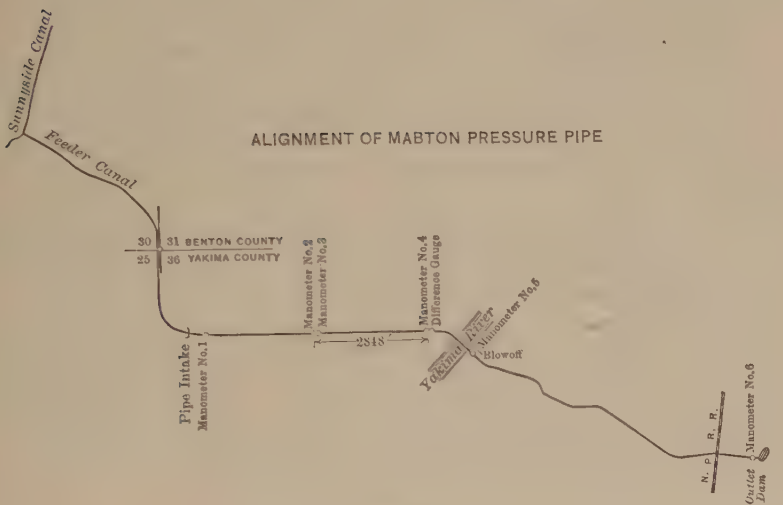

ALIGNMENT OF MABTON PRESSURE PIPE
Sunnyside Canal
Feeder Canal
30
25
31 BENTON COUNTY
36 YAKIMA COUNTY
Pipe Intake
Manometer No.1
Manometer No.2
Manometer No.3
2848
Manometer No.4
Difference Gauge
Yakima River
Manometer No.5
Blowoff
N. P. R. R.
Outlet
Dam
Manometer No.6

FIG. 7.

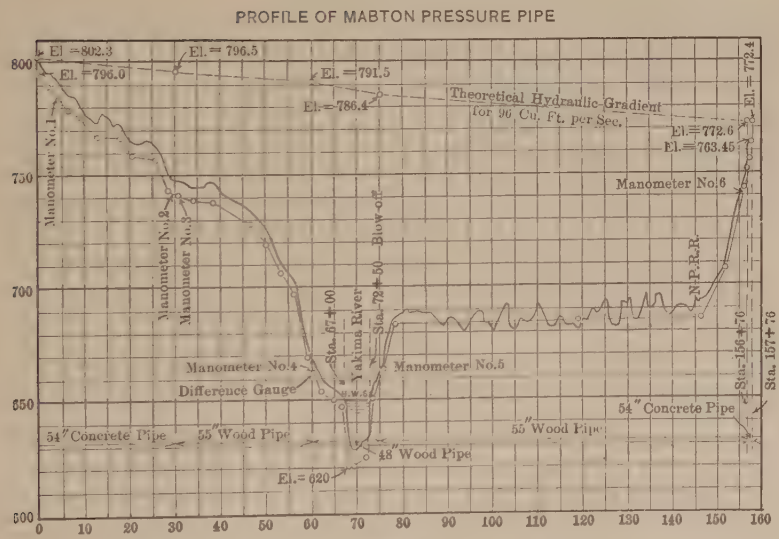

PROFILE OF MABTON PRESSURE PIPE
El. = 802.3
El. = 796.5
El. = 796.0
El. = 791.5
El. = 786.4
Theoretical Hydraulic Gradient
for 96 Cu. Ft. per Sec.
El. = 772.4
El. = 772.6
El. = 763.45
Manometer No.1
Manometer No.2
Manometer No.3
Manometer No.6
Sta. 67+00
Yakima River
Sta. 72+50 Blow-off
N.P.R.R.
Manometer No.4
Difference Gauge
Manometer No.5
Sta. 154+76
Sta. 157+76
54" Concrete Pipe
55" Wood Pipe
48" Wood Pipe
55" Wood Pipe
54" Concrete Pipe
El. = 620
800
750
700
650
600
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
160

FIG. 8.

TABLE 2.—Observed and Calculated Data

No.	Date.	Head on 18-ft. weir.	Discharge from rating table, c. f. s.	Mercury Gauge No. 8.									
				C.	U.	L.	Mercury.	T° cent.	t° cent.	Mercury reduc.	Water equivalent.	Elev. of "L" zero.	Elev. of water column.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
	1909												
1	Oct. 5	0.8550	53.40	1.443	0.590	0.195	1.838	15	11	1.8370	25.141	747.368	772.509
2	Oct. 5	0.8315	51.05	1.443	0.515	0.195	1.767	21	15	1.7650	24.164	747.368	771.532
3	Oct. 5	0.7580	44.32	1.443	0.483	0.195	1.731	17	11	1.7290	23.675	747.368	771.043
4	Oct. 6	0.6825	37.60	1.443	0.435	0.195	1.683	20	11	1.6805	23.016	747.368	770.384
5	Oct. 6	0.5975	30.80	1.443	0.410	0.195	1.658	25	13	1.6545	22.663	747.368	770.031
	1910												
1	Oct. 25	0.8520	52.10	1.093	0.879	0.102	1.870	27	14.5	1.866	25.442	749.066	774.508
2	Oct. 25	0.9850	66.85	1.093	0.896	0.102	1.887	22	15	1.885	25.700	749.066	774.766
3	Oct. 26	0.9620	64.32	1.093	0.849	0.102	1.840	5	6	1.841	25.103	749.066	774.169
4	Oct. 26	0.9075	57.98	1.093	1.001	0.102	1.992	13	11	1.992	27.153	749.066	776.219
5	Oct. 26	0.7200	40.90	1.093	0.774	0.102	1.765	19	13	1.763	24.044	749.066	773.110
6	Oct. 26	0.8080	48.82	1.093	0.809	0.102	1.800	12	10	1.800	24.546	749.066	773.612
7	Nov. 1	0.6055	31.44	0.880	0.791	0.103	1.568	17	13	1.567	21.383	749.055	770.438
8	Nov. 2	0.5300	25.70	0.880	0.662	0.103	1.439	10	5.5	1.438	19.641	749.055	768.696
9	Nov. 2	0.4695	21.46	0.880	0.480	0.103	1.207	13	7	1.206	16.480	749.055	765.535

operation was about 9 cu. ft. per sec., corresponding to a velocity of 3.5 ft. per sec., but, at the time the tests were made, it was not possible to make provision for carrying more than 8 cu. ft. per sec.

No recorded data are available in regard to the alignment or profile. Observations in the field indicated that the alignment consisted of two tangents making an angle of about 5 degrees. The pipe drops about 75 ft. below the hydraulic gradient at its lowest point, and there it has quite a sharp vertical curve. Levels run over the approximate location of the pipe indicate no summits in the ground, and, as it was supposed to be buried uniformly 2 ft. below the surface, it is probable that it has no summits. Four irrigation hydrants are attached to the pipe, and these may have some effect in roughening the interior surface, but as to this there is no definite knowledge. Nothing is known as to conditions at the time of construction. Measurements of the diameter at the intake and outlet gave an average of 22 in. The pipe seems to have been warped out of shape about 1 in.

Water columns were used for both gauges. The quantities for Observations 1 and 2 were measured over an 8-ft. round-crested weir, those for the remaining observations were measured over a 4-ft. sharp-crested weir. There were seven small leaks, approximately

ON 55¾-INCH CONTINUOUS WOOD STAVE PIPE.

Mercury Gauge No. 4.									Loss of Head.		Area.	Velocity.	*n*.
C.	U.	L.	Mercury.	T° cent.	t° cent.	Mercury reduc.	Water equivalent.	Elev. of "L" zero.	Elev. of water column.	Length, 2 848.2 ft.	Per 1 000 ft.			
(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)	(24)	(25)	(26)	(27)	(28)	(29)
6.686	0.608	0.200	7.154	15	11	7.150	97.297	674.130	771.427	1.082	0.380	16.98	3.145	0.01051
6.686	0.606	0.200	7.092	22	15	7.083	96.387	674.130	770.517	1.015	0.356	16.98	3.006	0.01062
6.686	0.588	0.200	7.074	18	11	7.066	96.156	674.130	770.286	0.759	0.266	16.98	2.610	0.01053
6.686	0.555	0.200	7.041	20	11	7.030	95.667	674.130	769.797	0.587	0.206	16.98	2.214	0.01083
6.686	0.545	0.200	7.031	25	13	7.016	95.477	674.130	769.607	0.424	0.149	16.98	1.814	0.01109
6.182	1.322	0.180	7.324	27	14.5	7.308	99.423	673.954	773.377	1.131	0.397	16.98	3.068	0.01098
6.182	1.291	0.180	7.293	22	15	7.284	99.097	673.954	773.051	1.715	0.602	16.98	3.937	0.01062
6.182	1.239	0.180	7.241	5	6	7.244	98.554	673.954	772.508	1.661	0.583	16.98	3.788	0.01084
6.182	1.416	0.180	7.418	13	11	7.416	100.889	673.954	774.843	1.376	0.483	16.98	3.415	0.01092
6.182	1.240	0.180	7.242	19	13	7.234	98.418	673.954	772.372	0.738	0.259	16.98	2.409	0.01118
6.182	1.252	0.180	7.254	12	10	7.252	98.662	673.954	772.616	0 996	0.350	16.98	2.875	0.01097
5.935	1.295	0.166	7.064	17	13	7.059	96.027	673.947	769.974	0.464	0.163	16.98	1.852	0.01137
5.935	1.176	0.166	6.945	10	5.5	6.941	94.425	673.947	768.372	0.324	0.114	16.98	1.513	0.01147
5.935	0.951	0.166	6.720	13	7	6.714	91.342	673.947	765.289	0.246	0.086	16.98	1.264	0.01173

within the first two-thirds of the distance between the gauges. These were measured volumetrically several times during the series of observations, and the correction to the measured quantity for this loss was assumed to be two-thirds of the total. In addition to these leaks between the gauges, a small leak was purposely maintained from the top of the pipe above Gauge No. 1 in order to release the small bubbles produced by the impact of the free flow down the pipe upon the main body of water under pressure. These small bubbles, unless released, rose slowly through the water column and vitiated the gauge readings.

There has been constant trouble with leaks on this pipe, due to defective material and workmanship, and frequent repairs have been made. It is entirely possible, therefore, that the interior surface is in bad condition; however, no definite information is available, as examination of the interior was impracticable. Table 3 gives the results of these experiments.

18-In. Jointed Wood Stave Pipe.—This pipe was built in 1908, and has been in operation two seasons. The normal velocity maintained during this time was about 3.7 ft. per sec. Quantities were measured over a 6-ft. Cippoletti weir at the outlet of the pipe. A leak in one of the gates in the outlet structure necessitated an additional

TABLE 3.—Observed and Calculated Data on 22-inch Jointed Wood Stave Pipe.

No.	Date, 1910.	Head on 8-ft. weir.	Head on 4-ft. weir.	Discharge, by Cippoletti formula, c. f. s.	Leakage between gauges.		Leakage above Gauge No. 1, c. f. s.	Total correction for leakage.	Net discharge, c. f. s.	Water gauge No. 1.			Water gauge No. 2.			Loss of head.		Area.	Velocity.	n.
					Total, c. f. s.	Correction, +c. f. s.				Average of readings.	Elevation of zero.	Elevation of water column.	Average of readings.	Elevation of zero.	Elevation of water column.	Total length, 2 087 ft.	Per 1 000 ft.			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)
1	Sept. 23	0.4345	.....	*7.864	0.0675	0.045	0.016	0.061	7.803	0.436	101.773	102.209	0.910	97.572	98.482	3.727	1.786	2.64	2.956	0.01263
2	Sept. 23	0.4510	.....	*8.320	0.0675	0.045	0.016	0.061	8.259	0.979	101.773	102.752	0.970	97.572	98.542	4.210	2.017	2.64	3.128	0.01267
3	Sept. 24	.....	0.4935	4.669	0.1260	0.084	0.016	0.100	4.569	1.581	97.487	99.068	0.470	97.572	98.042	1.026	0.492	2.64	1.731	0.01136
4	Sept. 27	.....	0.5050	4.830	0.1570	0.105	0.016	0.121	4.709	1.293	101.743	103.036	1.124	100.626	101.750	1.286	0.616	2.64	1.784	0.01220
5	Sept. 27	.....	0.4227	3.705	0.1570	0.105	0.015	0.120	3.585	1.505	101.743	103.248	0.892	101.630	102.522	0.726	0.348	2.64	1.358	0.01192
6	Sept. 27	.....	0.3570	2.874	0.1570	0.105	0.016	0.121	2.753	1.300	101.743	103.043	1.049	101.630	102.679	0.364	0.174	2.64	1.043	0.01084
7	Sept. 27	.....	0.2740	1.934	0.1570	0.105	0.016	0.121	1.813	1.585	101.743	103.328	0.457	102.724	103.181	0.147	0.070	2.64	0.687	0.00988

*The discharge by Cippoletti formula is increased 2% to correct for the use of a round-crested weir.

†The value for leakage correction between gauges is assumed to be two-thirds of the total leakage occurring in the first two-thirds of the length.

measurement of quantity over a 1-ft. weir. The alignment includes one light curve. The alignment, and the profile of the surface over the pipe are shown in Fig. 9. The top of the pipe is generally 18 in. below the surface, except at the summits, where it is deeper. Measurements of the diameter at the outlet gave an average of 18 in., the pipe being distorted about ½ in. at this point. The intake has a funnel attachment, thus making a measurement of the diameter at this point impracticable.

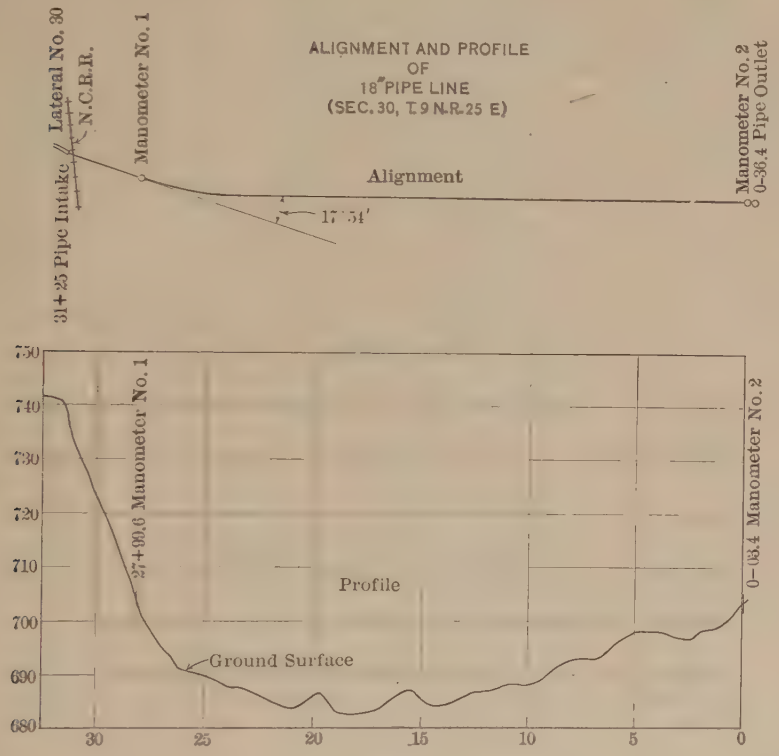


FIG. 9.

Water columns were used at both instruments. As previously stated, Gauge No. 1 was connected to the side of the pipe, the accumulation of air not permitting an attachment to the top. Though Gauge No. 2 was connected directly to the top of the pipe, it exhibited no signs of air disturbance at any time during the observations. For this reason it is hardly possible that any air was carried through the

TABLE 4.—Observed and Calculated Data on 18-Inch Jointed Wood Stave Pipe.

No.	Date, 1910.	Head on 6-ft. weir.	Discharge by Cippoletti formula, c. f. s.	Head on 1-ft. weir.	Discharge by Cippoletti formula, c. f. s.	Total disch. c. f. s.	Water Gauge No. 1.			Water Gauge No. 2. Elev. of zero, 98.271.		Loss of head.	Length of pipe.	Loss per 1 000 ft.	Area.	Velocity.	n.
							Aver. of readings.	Elev. of zero.	Elev. of water column.	Aver. of readings.	Elev. of water column.						
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)
1	Oct. 11	0.4450	6.000	0.052	0.040	6.040	1.179	103.541	104.720	1.122	99.393	5.327	2 802.5	1.901	1.767	3.418	0.01023
2	Oct. 12	0.3920	4.958	0.052	0.040	4.998	0.555	102.527	103.082	1.058	99.329	3.753	2 802.5	1.339	1.767	2.829	0.01032
3	Oct. 12	0.3304	3.837	0.052	0.040	3.877	0.512	101.289	101.801	1.001	99.272	2.529	2 802.5	0.902	1.767	2.194	0.01076
4	Oct. 13	0.2448	2.452	0.031	0.019	2.471	0.473	99.750	100.223	0.893	99.164	1.059	2 774.2	0.382	1.767	1.398	0.01076
5	Oct. 13	0.1948	1.737	0.031	0.019	1.756	1.180	98.522	99.702	0.842	99.113	0.589	2 774.2	0.212	1.767	0.994	0.01093
6	Oct. 13	0.1719	1.443	0.031	0.019	1.462	1.002	98.522	99.524	0.824	99.095	0.429	2 774.2	0.155	1.767	0.827	0.01098
7	Oct. 13	0.1568	1.252	0.031	0.019	1.271	0.909	98.522	99.431	0.810	99.081	0.350	2 774.2	0.126	1.767	0.719	0.01115
8	Oct. 13	0.1290	0.939	0.031	0.019	0.958	0.735	98.522	99.257	0.782	99.053	0.204	2 774.2	0.074	1.767	0.542	0.01080

entire length of the pipe. From the regular intervals at which the most violent fluctuations took place, it seemed as though the air was carried only a short distance below Gauge No. 1 and remained there until a sufficient quantity had collected to force itself back up the pipe. Table 4 gives the results of the experiments; and Fig. 13 shows the relation of n to the velocity.

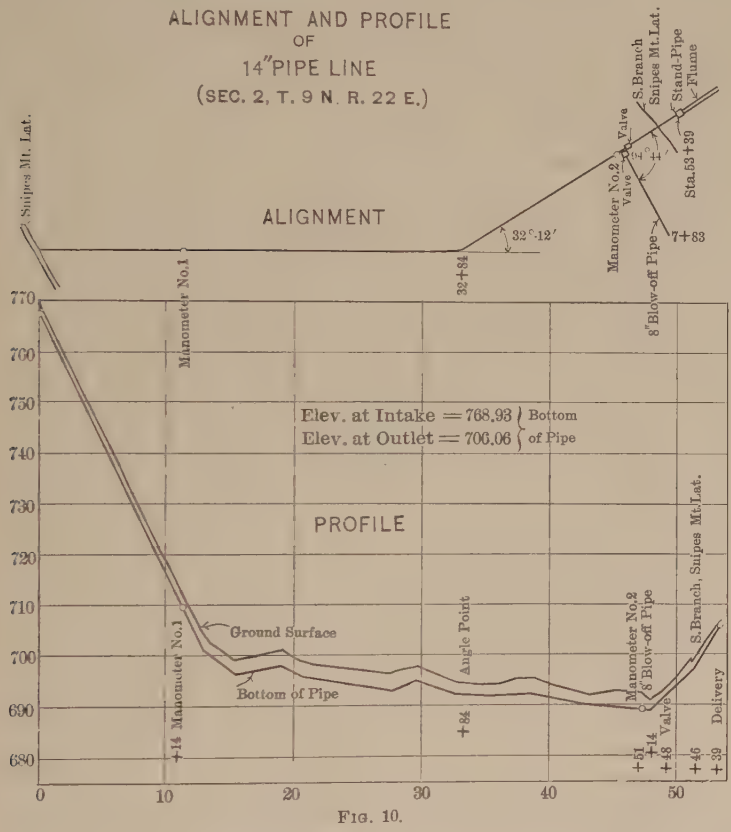


FIG. 10.

14-In. Jointed Wood Stave Pipe.—This pipe was built in 1905, and has been in use five seasons. Measurements at the intake showed an average diameter of 14 in. During the experiments a piece of rotten stave was blown out of the top by the increased pressure, and this afforded an opening through which an additional measurement of the diameter was taken. This also showed an average of 14 in., the distortion being about $\frac{1}{4}$ in. There was no growth of any kind on the

interior surface at this point, but there was evidence of considerable wear on the staves. The softer portions of the fir lumber had been worn down, leaving the harder parts to form small ridges approximately parallel to the axis of the pipe. This was the only place affording access to the interior, therefore it is not known that this condition obtained at other points. In any case, its effect on the friction losses would be difficult to predict. The approximate alignment and profile are shown in Fig. 10. The bend of 32° 12′ is made with a series of short sections of pipe, and is not as sharp as indicated in the diagram. The line representing the bottom of the pipe is conjectural for most of the distance, the only known points being one near the intake and one at each manometer. It is possible that the three summits shown do not exist, but, aside from this, the grade could not differ much. It will be noted that the manometers were located very favorably, as they were entirely free from any effect of either vertical or horizontal curvature.

Experiments were made on this pipe in 1909 and 1910, both sets on the same lengths, and with the gauges at the same points. Mercury gauges only were used. In 1909 the quantities were measured over a round-crested weir at the intake, the heads over the weir being taken with a 2-ft. rule. There was considerable disturbance in the weir box for the larger quantities, and before the experiments were begun in 1910 a better baffle-wall was inserted and a sharp crest placed on the weir. The heads were also measured with a hook-gauge in a stilling box. There was a small leak just above Gauge No. 1. This was measured volumetrically several times during the tests, and found to be almost constant at 0.03 cu. ft. per sec. This quantity is subtracted from the discharge measured at the intake. Table 5 gives the results of these experiments. The losses found in 1910 are smaller by about 3% than those found in 1909. The reasons for this can only be conjectured. It is possible that the pipe had become smoother, and it is also possible that some of the difference, if not all, is due to errors of observation. If the latter is true, then more weight should attach to the 1910 experiments, as the facilities for accurate measurements for these tests were better than for those of 1909. It is not probable that there was any silt deposit in the pipe at any time, as the normal velocity during the time of operation was about 2.75 ft. per sec.

12-In. Jointed Wood Stave Pipe.—This pipe was built in 1910,

TABLE 5.—Observed and Calculated Data on 14-Inch Jointed Wood Stave Pipe.

No.	Date.	Head on 2-ft. weir.	Discharge, in cubic feet per second.	Mercury Gauge No. 1.							Mercury Gauge No. 2.							Loss of Head.		Area.	Velocity.	n.
				C.	U.	L.	Mercury.	Water equivalent.	Elev. of "L" zero.	Elev. of water col.	C.	U.	L.	Mercury.	Water equivalent.	Elev. of "L" zero.	Elev. of water col.	Length, 3 637 ft.	Per 1 000 ft.			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)
	1909																					
1	Oct. 2.	0.1740	0.529*	1.053	0.164	0.073	1.144	15.609	98.036	113.645	1.051	1.664	0.248	2.467	33.750	79.511	113.261	0.384	0.106	1.069	0.495	0.01180
2	Oct. 7.	0.6170	3.493*	1.053	0.160	0.073	1.140	15.554	98.036	113.590	1.051	1.000	0.248	1.803	24.733	79.511	104.244	9.346	2.570	1.069	3.267	0.01038
3	Oct. 7.	0.5400	2.820*	1.053	0.142	0.073	1.122	15.310	98.036	113.346	1.051	1.159	0.248	1.962	26.892	79.511	106.403	6.943	1.909	1.069	2.638	0.01091
4	Oct. 7.	0.4635	2.224*	1.053	0.229	0.073	1.209	16.491	98.036	114.527	1.051	1.426	0.248	2.229	30.518	79.511	110.029	4.498	1.237	1.069	2.080	0.01105
5	Oct. 7.	0.3260	1.752*	1.053	0.391	0.073	1.371	18.691	98.036	116.727	1.051	1.713	0.247	2.517	34.428	79.511	113.939	2.788	0.767	1.069	1.639	0.01097
6	Oct. 8.	0.3230	1.298*	1.053	0.388	0.073	1.818	17.971	98.036	116.007	1.051	1.738	0.247	2.542	34.767	79.511	114.278	1.729	0.475	1.069	1.214	0.01138
7	Oct. 8.	0.2810	1.050*	1.053	0.288	0.073	1.268	17.292	98.036	115.328	1.051	1.735	0.247	2.539	34.727	79.511	114.238	1.090	0.300	1.069	0.982	0.01106
8	Oct. 8.	0.1800	0.555*	1.053	0.277	0.073	1.257	17.143	98.036	115.179	1.051	1.768	0.241	2.578	35.250	79.511	114.761	0.418	0.115	1.069	0.519	0.01182
	1910																					
1	Aug. 25.	0.4705	2.144+	1.039	0.332	0.076	1.295	17.662	98.477	116.189	1.882	0.658	0.076	2.464	33.537	78.598	112.135	4.004	1.101	1.069	2.006	0.01084
2	Aug. 25.	0.4135	1.761+	1.039	0.380	0.076	1.343	18.314	98.477	116.791	1.882	0.799	0.076	2.605	35.452	78.598	114.050	2.741	0.754	1.069	1.647	0.01085
3	Aug. 26.	0.3540	1.384+	1.039	0.488	0.076	1.451	19.781	98.477	118.258	1.882	0.981	0.076	2.787	37.923	78.598	116.521	1.737	0.478	1.069	1.295	0.01085
4	Aug. 26.	0.2950	1.050+	1.039	0.511	0.076	1.474	20.093	98.477	118.570	1.882	1.056	0.076	2.862	38.942	78.598	117.540	1.030	0.283	1.069	0.982	0.01079
5	Aug. 26.	0.2460	0.790+	1.039	0.510	0.076	1.473	20.079	98.477	118.556	1.882	1.085	0.076	2.891	39.336	78.598	117.934	0.622	0.171	1.069	0.739	0.01079
6	Aug. 27.	0.2085	0.612+	1.039	0.345	0.076	1.308	17.889	98.477	116.316	1.882	0.988	0.076	2.744	37.340	78.598	115.938	0.378	0.104	1.069	0.572	0.01046
7	Aug. 29.	0.5640	2.822+	1.032	0.449	0.076	1.405	19.156	98.477	117.633	1.894	0.565	0.076	2.383	32.437	78.598	111.035	6.598	1.814	1.069	2.640	0.01068
8	Sept. 6.	0.4690	2.133+	1.032	0.341	0.076	1.297	17.689	98.477	116.166	1.894	0.657	0.076	2.475	33.686	78.598	112.284	3.882	1.067	1.069	1.995	0.01075

* These quantities are derived from results of volumetric tests by Stevens.

† This discharge is by the Cippoletti formula, and is reduced by 0.030 cu. ft. per sec. to allow for a leak above Gauge No. 1.

and had been in use about 3 months at the time the observations were made. It was built to supplement the 22-in. pipe already described, and parallels it. The vertical curve at the low point is not as sharp as that of the 22-in. pipe. Measurements of discharge were made over a sharp-crested Cippoletti weir at the intake. A valve at the outlet made it possible to keep the head always above the top of the pipe at the intake, thus eliminating any trouble from air in suspension at the instruments. No leaks were observed at any point. Measurements of diameter at the intake gave an average of 12 in., and no distortion was apparent. A water column was used at Gauge No. 1 and a mercury column at Gauge No. 2. The normal velocity, up to the time of the tests, was about 0.8 ft. per sec.

Table 6 shows the results of the experiments. The indicated losses are abnormally high, as compared with the other pipes tested. There is no reason to suppose that this is due to abnormal roughness of the interior surface, although this is a new pipe and may become smoother. The magnitude of the losses and the nature of their variation, in the writer's opinion, do not indicate a rough interior. The writer would rather offer another explanation: The water flowing in this pipe carries quite a large quantity of silt in suspension, and the velocity under which the pipe had been operated up to the time the experiments were made was about 0.8 ft. per sec. It is known from experience that, even at higher velocities than this, such suspended material will deposit, and it would require no large quantity, in a pipe of this size, to reduce the area and correspondingly increase the calculated velocities to such an extent as to affect the apparent relation between velocities and losses of head quite materially. The writer hopes to obtain definite information on this point at an early date.

8-In. Jointed Wood Stave Pipe.—This pipe was built in the spring of 1909, and had been in use about 5 months before the first experiments. Experiments were also made in 1910. In 1909 the observations were taken on lengths of 3 395 and 2 002 ft. A water column was used at the outlet and a mercury column at the other stations. For the 1910 experiments the water column near the outlet was placed in the same position it had occupied in the 1909 tests. The mercury gauge for the first eight observations was placed about 120 ft. above the old location, making the testing length 3 515.2 ft., and for the remaining observations a water gauge was substituted, and the distance between the gauges was increased to 4 054.9 ft.

TABLE 6.—Observed and Calculated Data on 12-Inch Jointed Wood Stave Pipe.

No.	Date, 1910.	Head on 4-ft. weir.	Discharge by Cippoletti formula, c. f. s.	Gauge No. 1—Water Gauge.			Gauge No. 2—Mercury Gauge. C=0.876 ft. Elev. of "L" zero, 81.947.					Loss of Head.		Area.	Velocity.	*n*.
				Average of readings.	Elevation of zero.	Elev. of water column.	U.	L.	Mercury.	Water equivalent.	Elev. of water col.	Length, 2 022 ft.	Per 1 000 ft.			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)
1	Oct. 8	0.0900	0.360	1.098	97.907	99.005	0.465	0.114	1.227	16.777	98.724	0.281	0.139	0.785	0.459	0.01290
2	Oct. 8	0.1065	0.469	1.238	97.907	99.145	0.463	0.114	1.225	16.750	98.697	0.448	0.222	0.785	0.597	0.01302
3	Oct. 8	0.1230	0.581	1.482	97.907	99.389	0.462	0.114	1.224	16.736	98.683	0.706	0.349	0.785	0.740	0.01404
4	Oct. 8	0.1385	0.698	1.515	97.907	99.422	0.448	0.114	1.210	16.546	98.493	0.929	0.459	0.785	0.889	0.01305
5	Oct. 10	0.1765	0.998	0.476	98.899	99.375	0.391	0.114	1.153	15.772	97.719	1.656	0.819	0.785	1.271	0.01255
6	Oct. 10	0.2000	1.200	0.925	98.899	99.824	0.368	0.114	1.130	15.459	97.406	2.418	1.196	0.785	1.529	0.01267
7	Oct. 10	0.2225	1.415	0.810	98.899	99.709	0.307	0.114	1.069	14.631	96.578	3.131	1.548	0.785	1.803	0.01235

TABLE 7.—Observed and Calculated Data

No	Date.	Meas't at Intake.		Measurement at Outlet.			Average discharge, c. f. s.	Gauge					
								Mercury					
		Head on 2-ft. weir.	Discharge, c. f. s.	Head on 3-ft. weir.	Head on 1-ft. weir.	Discharge, c. f. s.		C.	U.	L.	Mercury.	Water equivalent.	Elev. of "L" zero.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
	1909												
1	Sept. 28	0.2640	0.961*	.....	.....	.....	.....	1.055	1.226	0.072	2.209	30.070	100.335
2	Sept. 28	0.1350	0.366*	.....	.....	.....	.....	1.055	0.199	0.072	1.182	16.124	100.335
3	Sept. 28	0.1600	0.469*	.....	.....	.....	.....	1.055	0.315	0.071	1.299	17.711	100.335
4	Sept. 28	0.1840	0.573*	.....	.....	.....	.....	1.055	0.455	0.071	1.439	19.613	100.335
5	Sept. 28	0.2215	0.745*	.....	.....	.....	.....	1.055	0.745	0.071	1.729	23.551	100.335
6	Sept. 29	0.2530	0.904*	.....	.....	.....	.....	1.055	1.111	0.071	2.095	28.521	100.335
7	Sept. 29	0.2800	1.045*	.....	.....	.....	.....	1.055	1.436	0.070	2.421	32.947	100.335
8	Sept. 29	0.3315	1.349*	.....	.....	.....	.....	2.064	1.244	0.065	3.243	44.105	100.335
9	Sept. 29	0.3630	1.538*	.....	.....	.....	.....	3.067	0.803	0.063	3.867	51.762	100.335
10	Oct. 27	0.1515	0.433*	.....	.....	.....	.....	1.105	0.794	0.236	1.663	22.820	80.033
11	Oct. 27	0.2290	0.782*	.....	.....	.....	.....	1.105	1.122	0.234	1.993	27.299	80.033
	1910												
1	Sept. 7	0.4480	2.018+	0.3490	.....	2.081+	2.050	2.950	1.058	0.077	3.931	53.460	101.392
2	Sept. 8	0.4140	1.794+	0.3180	.....	1.812+	1.803	2.950	0.455	0.077	3.328	45.271	101.392
3	Sept. 8	0.3660	1.492+	0.2770	.....	1.476+	1.484	1.916	0.740	0.077	2.579	35.100	101.392
4	Sept. 8	0.3400	1.330+	0.2550	.....	1.306+	1.315	1.916	0.463	0.076	2.303	31.351	101.392
5	Sept. 9	0.3120	1.172+	0.2370	.....	1.166+	1.169	1.027	1.087	0.076	2.038	27.752	101.392
6	Sept. 9	0.2665	0.922+	0.1980	.....	0.888+	0.905	1.027	0.693	0.076	1.644	22.402	101.392
7	Sept. 9	0.2340	0.760+	0.1745	.....	0.737+	0.748	1.027	0.476	0.076	1.427	19.455	101.392
8	Sept. 9	0.1960	0.584+	0.1460	.....	0.566+	0.575	1.027	0.281	0.076	1.232	16.807	101.392
9	Sept. 9	0.1590	0.426+	0.1200	.....	0.420+	0.423	.....	.....	.....	.....	.....	101.392
10	Sept. 10	0.1320	0.326+	.....	0.216	0.338+	0.332	.....	.....	.....	.....	.....	101.392
11	Sept. 10	0.0910	0.183+	.....	0.148	0.182+	0.182	.....	.....	.....	.....	.....	101.392
12	Sept. 10	0.0810	0.153+	.....	0.133	0.163+	0.158	.....	.....	.....	.....	.....	101.392

* These quantities are derived from the results of volumetric tests by Stevens.

Fig. 11 shows the approximate alignment and profile of the pipe, and the favorable location of the manometers. The bend of 16° 40′ is made with a number of short sections, in the same manner as that for the 14-in. pipe, and is not as sharp as indicated. The only known elevations of the pipe are at the manometers and at the intake and outlet, the remainder being estimated.

The discharge in the 1909 experiments was measured at the intake, over a 2-ft. round-crested Cippoletti weir. In 1910 a sharp crest was placed on this weir, and simultaneous measurements were made at the outlet over a 3-ft. sharp-crested weir. For the last three observations, when the head over the 3-ft. weir became too small to afford accurate results, a 1-ft. weir was substituted. All heads were measured with hook-gauges. The value of the discharge used in the reductions is the mean of the quantities measured at the intake and outlet.

ON 8-INCH JOINTED WOOD STAVE PIPE.

No. 1.				Gauge No. 2.								
gauge.	Water gauge.			Water gauge.								
Elev. of water col.	Aver. of readings.	Elev. of zero.	Elev. of water col.	Aver. of readings.	Elev. of "L" zero.	Elev. of water col.	Loss of head.	Length of pipe	Loss per 1 000 ft.	Area.	Velocity.	n.
(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)	(24)	(25)	(26)	(27)
130.405	.....	.....	.....	0.935	112.925	113.860	16.545	3394.5	4.874	0.349	2.754	0.01101
116.459	.....	.....	.....	0.658	112.925	113.583	2.876	3394.5	0.847	0.349	1.049	0.01159
118.046	.....	.....	.....	0.694	112.925	113.619	4.427	3394.5	1.304	0.349	1.344	0.01140
119.948	.....	.....	.....	0.733	112.925	113.658	6.290	3394.5	1.853	0.349	1.642	0.01123
123.896	.....	.....	.....	0.817	112.925	113.742	10.144	3394.5	2.988	0.349	2.135	0.01107
128.856	.....	.....	.....	0.899	112.925	113.824	15.032	3394.5	4.428	0.349	2.590	0.01112
133.282	.....	.....	.....	0.985	112.925	113.910	19.372	3394.5	5.707	0.349	2.994	0.01098
144.440	.....	.....	.....	1.189	112.925	114.114	30.326	3394.5	8.934	0.349	3.866	0.01074
152.097	.....	.....	.....	1.328	112.925	114.253	37.844	3394.5	11.149	0.349	4.407	0.01057
102.853	.....	.....	.....	0.618	100.004	100.622	2.231	2002.0	1.114	0.349	1.241	0.01138
107.332	.....	.....	.....	0.782	100.004	100.786	6.546	2002.0	3.270	0.349	2.241	0.01105
154.852	.....	.....	.....	1.551	112.896	114.447	40.405	3515.2	11.494	0.349	5.874	0.00859
146.663	.....	.....	.....	1.428	112.896	114.324	32.339	3515.2	9.200	0.349	5.166	0.00871
136.492	.....	.....	.....	1.251	112.896	114.147	22.345	3515.2	6.357	0.349	4.252	0.00877
132.743	.....	.....	.....	1.180	112.896	114.076	18.667	3515.2	5.310	0.349	3.768	0.00897
129.144	.....	.....	.....	1.117	112.896	114.013	15.131	3515.2	4.305	0.349	3.350	0.00905
123.794	.....	.....	.....	1.011	112.896	113.907	9.887	3515.2	2.813	0.349	2.593	0.00933
120.847	.....	.....	.....	0.946	112.896	113.842	7.005	3515.2	1.993	0.349	2.143	0.00945
118.199	.....	.....	.....	0.883	112.896	113.779	4.420	3515.2	1.257	0.349	1.648	0.00963
.....	0.715	116.183	116.898	0.833	112.896	113.729	3.169	4054.9	0.782	0.349	1.212	0.01008
.....	0.757	115.174	115.931	0.899	112.896	113.795	2.136	4054.9	0.527	0.349	0.951	0.01033
.....	1.440	113.100	114.540	0.810	112.896	113.706	0.834	4054.9	0.206	0.349	0.522	0.01093
.....	1.243	113.100	114.343	0.795	112.896	113.691	0.652	4054.9	0.161	0.349	0.453	0.01090

† By Cippoletti formula.

Table 7 gives the results of the experiments. Fig. 14 shows the variation of n with the velocity.

Some peculiar conditions were brought out during these experiments: When the pipe was first "tried out" in 1909, preliminary to making the experiments, it was impossible to get a larger quantity than 0.96 cu. ft. per sec. through it, even with a head of 5 ft. at the intake. It seemed certain that something must be wrong, as the probable flow under the conditions had been roughly estimated at between 1.5 and 2 cu. ft. per sec. Examination showed that air imprisoned in the pipe was causing the difficulty. This was overcome by inserting a ½-in. wrought-iron stand-pipe in the top of the pipe about 15 ft. below the intake. In this way the air was, to all appearances, entirely removed, and the carrying capacity was raised to 1.54 cu. ft. per sec.—an increase of about 60 per cent. In 1910

it was found that the pipe would carry 2.02 cu. ft. per sec. with practically the same loss of head—an increase of 31% over the maximum carried in 1909, and an increase of 110% over the quantity carried before the air was released.

The great increase in carrying capacity in 1910 over that found in 1909 is difficult to explain. The writer, at least, can offer no explana-

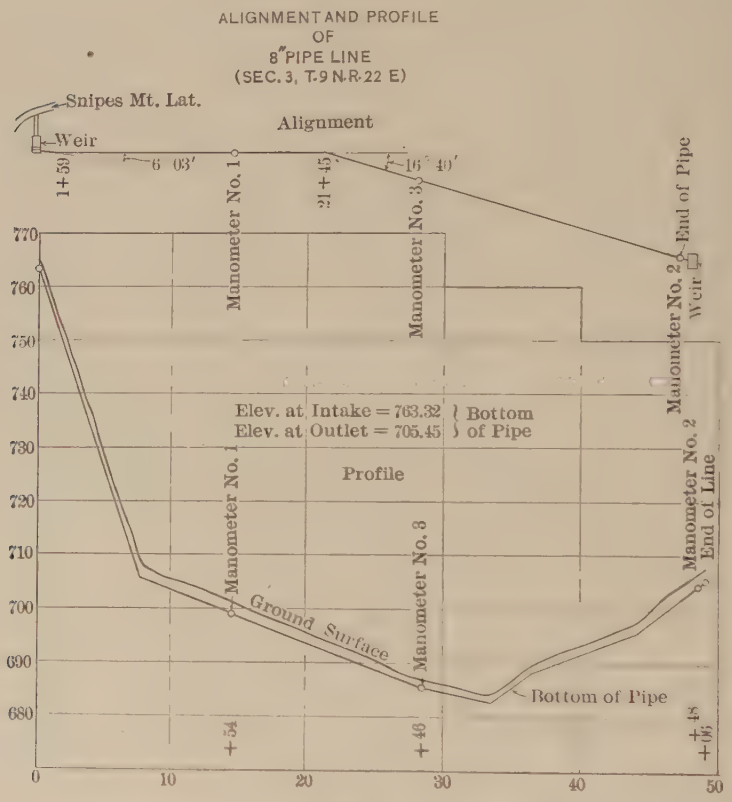


FIG. 11.

tion, unless it be that there was still some air under pressure in the pipe during the 1909 experiments, after the stand-pipe was attached, and that this had the effect of increasing the losses to the extent indicated. This would appear to be the most plausible explanation, even after taking into account the fact that no additional air valves were inserted in 1910, the pipe having been operated under identical

conditions. The pipe was used for about 6 months in the interval between the two sets of experiments, but it is hardly possible that it had worn so much smoother during this time, even if greater smoothness could have produced the effect indicated, which the writer does not believe to be the case. The presence of silt in the pipe cannot be offered as an explanation, as the velocities, ever since the pipe was put in operation, have been too high to permit the formation of any deposits. The writer also took other means of assuring himself that such a condition did not exist.

6-In. Jointed Wood Stave Pipe (1909).—This pipe was built in the spring of 1909, and had been in use about 5 months before the experiments were made. The pipe is only 1 000 ft. long, and the alignment consists of a single tangent. It runs under a very uniformly sloping ground surface, and only very light vertical curves are possible. No record of the profile of the pipe is available. The maximum length on which it was practicable to experiment was 500 ft. Water columns were used at both gauges, and the discharges were measured over a 12-in. Cippoletti weir for which the method of reduction has been described. The range of velocities was limited by conditions at the intake. Table 8 gives the results.

6-In. Jointed Wood Stave Pipe (1910).—This pipe has a total length of 4 000 ft., but, owing to the fact that there are several hydrants in the second half of its length, observations were made on only 1 801 ft. The pipe was built in the spring of 1910, and had been operated about 4 months before the experiments were made. The normal velocity during this time was about 3 ft. per sec.

The alignment and profile are very similar to those of the 8-in. pipe, and the manometers were favorably located as regards curvature, etc. There appeared to be a slight summit about 500 ft. above the second manometer. Water gauges were used at both stations, and the discharge was measured at the intake over a 12-in. Cippoletti weir. The results of the 1910 observations are also given by Table 8.

5-In. Jointed Wood Stave Pipe.—No positive information was obtained as to when this pipe was built, but it was probably about 2 years old at the time of the experiments. It has a total length of 3 000 ft., and observations were taken over a length of 1 822 ft. The diameter, as nearly as could be determined, is 5 in. The pipe seemed to be in good condition. It crosses a wide, shallow hollow, and runs

TABLE 8.—OBSERVED AND CALCULATED DATA ON 6-INCH JOINTED WOOD STAVE PIPE.

No.	Date.	Head on 1-ft. weir.	Discharge, c.f.s.	Water Gauge No. 1.			Water Gauge No. 2.			Loss of head.	Length of pipe.	Loss per 1 000 ft.	Area	Velocity.	n
				Aver. of readings.	Elev. of zero.	Elev. of water column.	Aver. of readings.	Elev. of zero.	Elev. of water column.						
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)
	1909														
1	Oct. 26	0.089*	0.0979†	1.448	92.475	93.923	1.438	92.360	93.798	0.125	500.4	0.250	0.196	0.499	0.01036
2	Oct. 26	0.1255*	0.1620†	0.761	92.475	93.236	0.571	92.360	92.931	0.305	500.4	0.610	0.196	0.827	0.01029
3	Oct. 26	0.151*	0.2127†	0.672	92.475	93.147	0.289	92.360	92.649	0.498	500.4	0.995	0.196	1.085	0.01020
4	Oct. 26	0.178*	0.2700†	1.386	92.475	93.861	0.714	92.360	93.074	0.787	500.4	1.573	0.196	1.378	0.01018
5	Oct. 26	0.2055*	0.3340†	1.323	92.475	93.798	0.302	92.360	92.662	1.136	500.4	2.270	0.196	1.704	0.01008
	1910														
1	Sept. 15	0.1305	0.159‡	0.428	102.773	103.201	0.999	101.111	102.110	1.091	1801	0.606	0.196	0.811	0.01041
2	Sept. 15	0.149	0.194‡	1.056	102.773	103.829	1.043	101.111	102.154	1.675	1801	0.930	0.196	0.990	0.01063
3	Sept. 15	0.1085	0.121‡	0.999	101.734	102.733	0.956	101.111	102.067	0.666	1801	0.370	0.196	0.617	0.01045
4	Sept. 15	0.093	0.096‡	0.737	101.734	102.471	0.925	101.111	102.036	0.435	1801	0.240	0.196	0.490	0.01032
5	Sept. 16	0.082	0.079‡	0.569	101.734	102.303	0.903	101.111	102.014	0.289	1801	0.160	0.196	0.403	0.01002
6	Sept. 16	0.177	0.251‡	0.876	104.980	105.856	0.192	103.159	103.351	2.505	1801	1.391	0.196	1.281	0.01026
7	Sept. 16	0.199	0.299‡	0.628	107.493	108.121	1.229	103.159	104.388	3.733	1892	1.973	0.196	1.526	0.01029
8	Sept. 17	0.3125	0.588‡	1.216	119.334	120.550	1.416	105.211	106.627	13.923	2021	6.889	0.196	3.000	0.00998
9	Sept. 17	0.264	0.456‡	0.650	112.751	113.401	0.064	105.211	105.275	8.126	1892	4.295	0.196	2.327	0.01009

*These heads were measured over a weir having a galvanized-iron crest.

†These discharges were derived from a comparison between a weir having a galvanized-iron crest and one having a sharp crest.

‡By Cippoletti formula.

under low pressure. No records are available as to its profile, except a few sights taken on the surface of the ground and a general observation of the surface, which would indicate a gradual fall and rise of the pipe grade. The alignment consists of a single tangent. The results are shown in Table 9.

The discharge was measured at the outlet over an 8-in. sharp-crested Cippoletti weir. Water columns were used for measuring all pressures at the outlet, Manometer No. 2, and for all but two observations at Manometer No. 1. A mercury column was necessary for Observations 3 and 4 on account of the increased pressures. The pipe carried considerable air in the first 1 000 ft., and it seemed to be practically impossible to get rid of it. For this purpose, four ½-in. taps were inserted in the top of the pipe in the first 600 ft., but without success. The manometers were placed outside of this zone, and air disturbances caused no particular trouble in taking the readings.

4-In. Jointed Wood Stave Pipe.—This pipe was built in 1907, and had been in service 3 years. The velocity of the water during this time was about 3.5 ft. per sec. It runs under a low head, on a continuous down grade to an outlet in a stand-pipe about 6 ft. high. The total length is 1 200 ft. The alignment consists of a single tangent. Table 10 gives the results of the observations, and Fig. 15 shows the variations of n with the velocity.

The discharge was measured over a 12-in. sharp-crested Cippoletti weir. It was found impossible to maintain an absolutely regular flow through the pipe, and the gauges, therefore, were somewhat unsteady. The averages obtained, however, are considered to represent the actual conditions within any practical limit of accuracy. Water columns were used for all gauges. For the two largest quantities, observations were taken on a length of 1 109.5 ft., but, for the smaller pressures, only 603 ft. could be utilized. The average diameter, as measured, was 4 in. The smaller discharges may be open to suspicion, on account of the low heads over the weir. The writer wishes that this condition had not obtained, but he is not ready to agree that any error from this source could be appreciable. This same objection might also be urged against the lower discharges on some of the other pipes.

At the intake, which point offered the only opportunity for an examination, it was noted that the interior surface was covered with a fungous growth, from $\frac{1}{8}$ to $\frac{3}{16}$ in. in thickness. It is not known

TABLE 9.—Observed and Calculated Data on 5-Inch Jointed Wood Stave Pipe.

No.	Date, 1909.	Head on 8-in. weir.	Discharge by Cippoletti formula, c. f. s.	Gauge No. 1.								Gauge No. 2.		Loss of Head.		Area.	Velocity.	n.
				Mercury gauge. C = 1.091 ft. Elev. of "L" zero, 99.687.					Water gauge.			Water gauge. Elev. of zero, 100.964.						
				U.	L.	Mercury.	Water equivalent.	Elev. of water col.	Aver. of readings.	Elev. of zero.	Elev. of water col.	Aver. of readings.	Elev. of water col.	Length, 1 822 ft.	Per 1 000 ft.			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)
1	Oct. 13	0.1540	0.136	.....	.....	.....	.....	.....	0.372	103.621	103.993	0.639	101.603	2.390	1.312	0.136	1.000	0.01082
2	Oct. 13	0.1855	0.179	.....	.....	.....	.....	.....	1.207	104.435	105.642	0.734	101.698	3.944	2.165	0.136	1.316	0.01069
3	Oct. 13	0.2690	0.313	0.152	0.246	0.997	13.785	113.472	.....	.....	.....	1.044	102.008	11.464	6.292	0.136	2.301	0.01055
4	Oct. 15	0.2680	0.311	0.125	0.246	0.970	13.419	113.106	.....	.....	.....	1.030	101.994	11.112	6.099	0.136	2.287	0.01048
5	Oct. 15	0.0860	0.057	.....	.....	.....	.....	.....	1.034	100.877	101.911	0.504	101.468	0.443	0.243	0.136	0.419	0.01041
6	Oct. 15	0.1165	0.089	.....	.....	.....	.....	.....	0.671	101.885	102.556	0.563	101.527	1.029	0.565	0.136	0.654	0.01066
7	Oct. 15	0.1345	0.111	.....	.....	.....	.....	.....	1.206	101.885	103.091	0.602	101.566	1.525	0.837	0.136	0.816	0.01057
8	Oct. 15	0.1990	0.199	.....	.....	.....	.....	.....	0.878	105.413	106.291	0.807	101.771	4.520	2.481	0.136	1.463	0.01042

TABLE 10.—Observed and Calculated Data on 4-Inch Jointed Wood Stave Pipe.

No.	Date, 1910.	Head on 1-ft. weir.	Discharge by Cippoletti formula, c. f. s.	Leakage above Gauge No. 1, c. f. s.	Net disch., c. f. s.	Water Gauge No. 1.			Water Gauge No. 2.			Loss of head.	Length of pipe.	Loss per 1 000 ft.	Area.	Velocity.	n
						Average of readings.	Elevation of zero.	Elevation of water column.	Average of readings.	Elevation of zero.	Elevation of water column.						
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)
1	Oct. 18	0.2370	0.388	.....	0.388	0.866	132.811	133.677	0.171	105.359	105.530	28.147	1 109.5	25.369	0.087	4.460	0.00956
2	Oct. 18	0.2050	0.312	.....	0.312	1.204	131.782	132.986	0.995	112.601	113.596	19.390	1 109.5	17.476	0.087	3.586	0.00977
3	Oct. 19	0.1290	0.156	0.010	0.146	0.925	106.839	107.764	0.452	104.354	104.806	2.958	608.0	4.905	0.087	1.678	0.01065
4	Oct. 19	0.1120	0.126	.....	0.126	1.363	106.839	108.202	0.593	105.385	105.978	2.224	608.0	3.688	0.087	1.448	0.01068
5	Oct. 20	0.0590	0.049	0.012	0.037	1.808	106.839	108.647	0.990	107.377	108.367	0.280	608.0	0.464	0.087	0.425	0.01178
6	Oct. 20	0.0980	0.096	0.005	0.091	0.758	106.839	107.597	0.885	105.380	106.265	1.332	608.0	2.209	0.087	1.046	0.01116
7	Oct. 20	0.0735	0.067	.....	0.067	1.074	106.839	107.913	0.757	106.397	107.154	0.759	608.0	1.259	0.087	0.770	0.01129

whether this condition exists throughout the length of the pipe, but, if it does, the area of the cross-section must be materially affected thereby, which would make the results quite different from those given in Table 10. The writer has had no opportunity to observe any growth of this nature, and is not prepared to offer an opinion as to the probability of its presence throughout the pipe. He would be glad to have some light on this subject.

Discussion of Results.

The diagram, Plate LVIII, contains a comprehensive plotting of the entire set of experiments. The scales are logarithmic, and were chosen because they afford the easiest means of examining the results, and because engineers have become accustomed to this method of analyzing experimental data. That the method is especially adapted to an analysis of experimental data on the flow of water in pipes requires no demonstration.

Before proceeding to an explanation of the diagram it should be stated that the writer has assumed that the loss of head in any wood stave pipe can be represented by an equation of the general form:

$$H = K\ D^z\ V^x;$$

where K, z, and x are constants to be determined by experiment;

H = the loss of head, in feet per 1 000 ft.;
D = the diameter of the pipe, in inches;
and V = the mean velocity of flow, in feet per second.

By assuming that $K\ D^z = m$, the formula can be broken into two equations, as follows:

$$m = K\ D^z \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

$$\text{and } H = m\ V^x \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

Both of these, being exponential equations, will plot as straight lines on logarithmic scales. The experimental data give the corresponding values of H and V, and, after plotting these for each pipe, the values of m and x can be readily determined, thus making the required constants known. In Plate LVIII the curves with positive slope represent this plotting. The appropriate scales are above and on the left of the diagram, representing "Loss of head per 1 000 ft." and "Velocity, in feet per second," respectively. The value of x is represented by the slope of the curve, and, as the horizontal and vertical scales of the diagram have the same spacing, this value is given

PLATE LVIII.
TRANS. AM. SOC. CIV. ENGRS.
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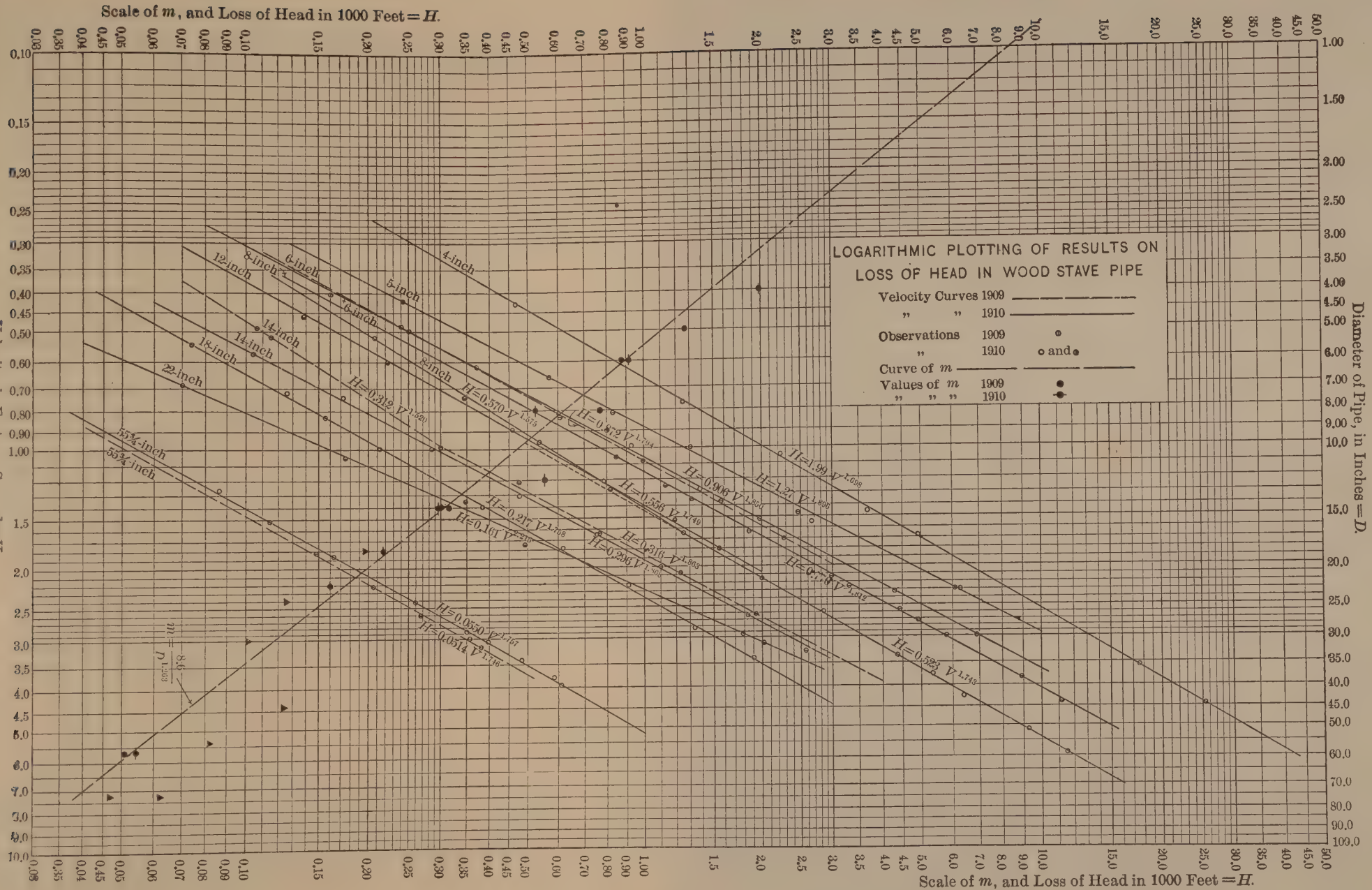
MORITZ ON
FLOW OF WATER IN WOOD PIPE.



directly by the tangent of the angle between the curve and the vertical axis. The value of m is represented by the intercept of the curve on the line for $V = 1$. The values thus obtained have been substituted in the general equation, $H = m\ V^x$, and the appropriate equation is written over each curve in the diagram.

In the equation, $m = K\ D^z$, we now have corresponding values of m and D, so that, by plotting these values, it is possible to determine K and z. The appropriate scales for this plotting are above and on the right of the diagram. It happens that the scale for values of m can also be used for values of H. It also happens that the scale of pipe diameters at the right is ten times the scale of velocities at the left, thus simplifying the diagram considerably. The points representing the relation between the values of m and x, found as above described, are the circular solid black ones. The dashed line with negative slope is intended to represent the average of these. The slope of this line is the value of z, and its intercept on the line for $D = 1$ is the value of K. The equation of this line is thus found to be $m = 8.6\ D^{-1.263}$.

Attention is called to the fact that for the 8-in. pipe (1910 series) and for the 14-in. pipe (1909 series) two slopes are given, a broken line being required to represent properly the locus of all the points. The writer has been unable to find an explanation for this phenomenon. It may be an indication of "critical velocity" conditions, but the data necessary to establish this point are too meager to warrant an analysis. The strange part of it is that this phenomenon did not develop on any of the other pipes. In determining the values of m, the higher slopes only have been used.

For the purpose of argument, the writer wishes to state that he has concluded that the following values of the constants best represent the average results of these experiments:

$$x = 1.8$$
$$z = -1.263$$
$$m = 8.6$$

Disregarding the lower slopes for the 8- and 14-in. pipes, it will be noted that the values of x vary from a minimum of 1.698 for the 4-in. pipe to a maximum of 2.215 for the 22-in. pipe. It will also be noted that the remaining values lie between 1.746 and 1.896, while most of them are between 1.746 and 1.85—not a large range, but sufficiently large to give quite wide variations in calculated quantities.

The average value, excluding the value of 2.215 for the 22-in. pipe, is 1.795. The writer would exclude this value, as being abnormal and far from representing average conditions. He also is quite firmly convinced that a thorough, scientific investigation of this same pipe would reveal the cause of the difficulty, and that it could be remedied without great trouble or expense.

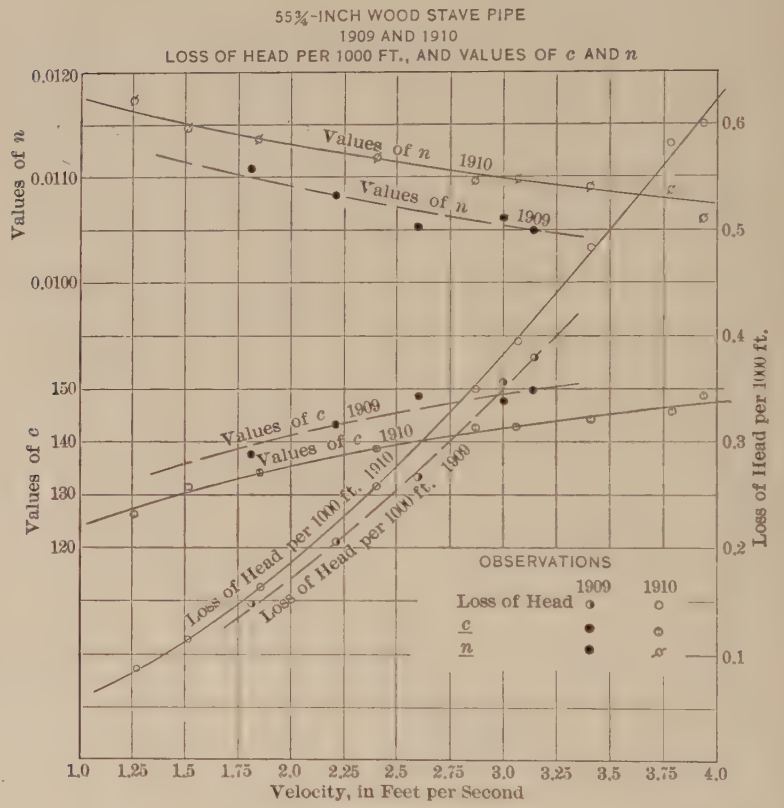


FIG. 12.

Pursuing this subject further, it might be inferred from previous statements that the 8-in. pipe of 1909 and the 12-in. pipe were operated under abnormal conditions, and that the values of x for these pipes should not be used in deducing the average. Leaving out these values, the average would be 1.798, as compared with 1.795—a difference not worthy of discussion—but this does not answer the

question, and the writer can offer no strenuous argument for not leaving the 8-in. pipe out of consideration. If left out, the effect on the average value of x is slight, and there does not seem to be sufficient reason for doing so. During the 1909 experiments the writer was under the impression that the pipe was operating under normal conditions, but some generous spirit evidently entered the field in the interim, and the 1910 experiments showed quite different results. Most of the difference, how-

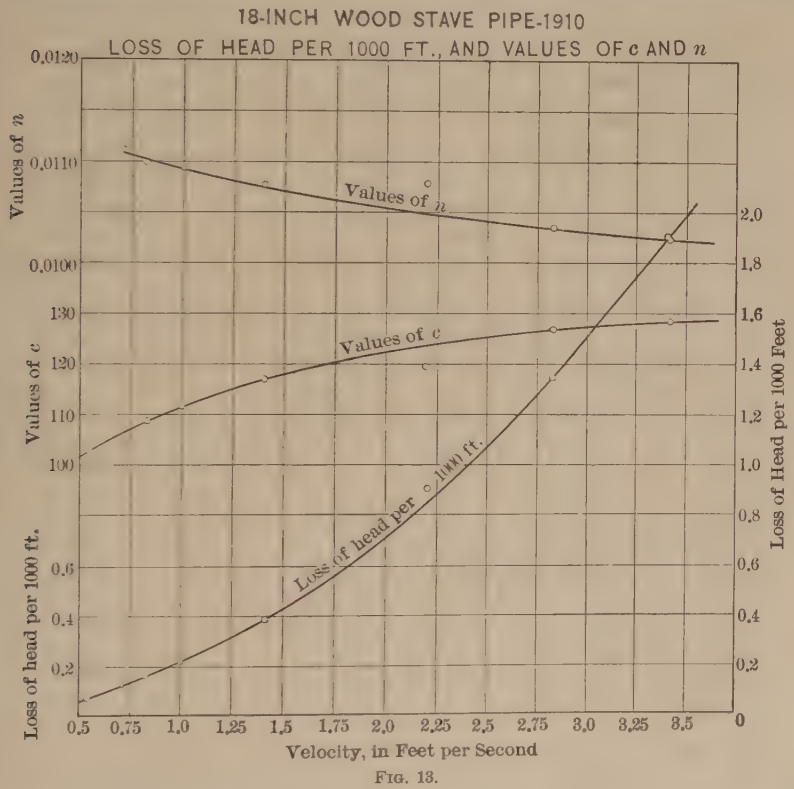


FIG. 13.

ever, is shown in the position of the curve rather than in its slope. In regard to the 12-in. pipe, the writer contends that the slope is normal, and that the nature of the abnormality, namely, the presence of silt in the pipe, is such as to affect, principally, the position of the curve and not its slope. This argument, of course, is not founded on an analysis of this pipe alone, but on a comparison with the entire series, which the writer considers is the only proper and safe method of deducing logical

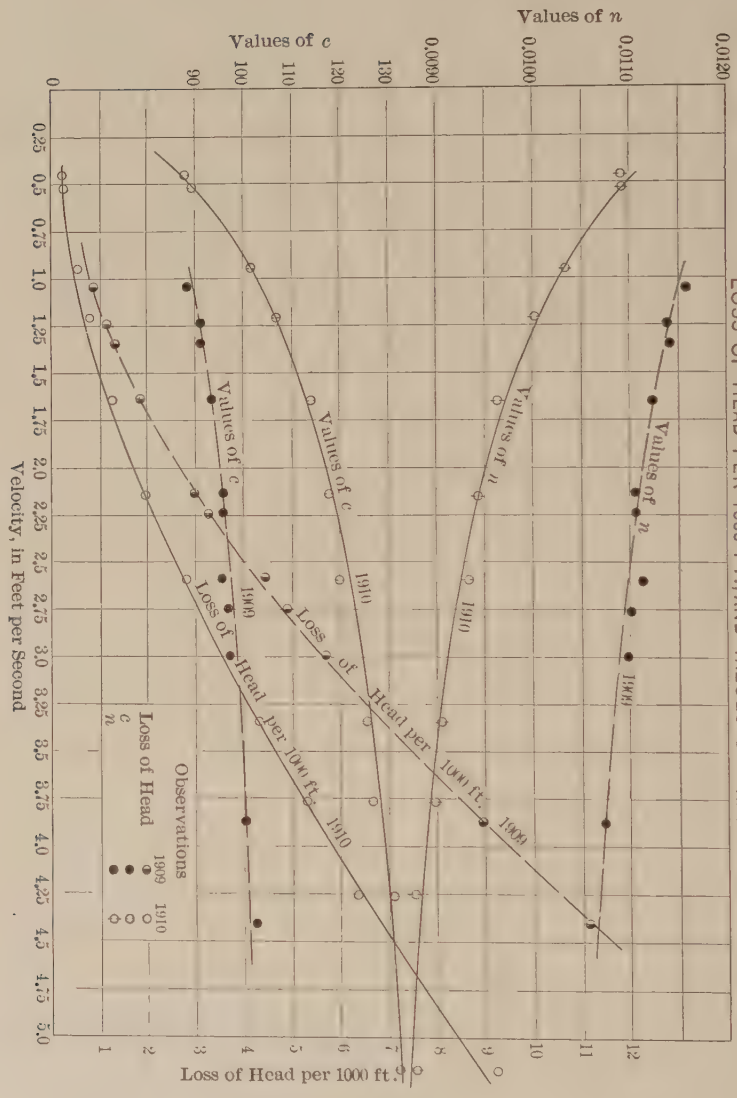

8-INCH WOOD STAVE PIPE, 1909 AND 1910
LOSS OF HEAD PER 1000 FT., AND VALUES OF c AND n
Values of n
Values of c
Values of n
Values of c
Values of c
Values of n
Loss of Head per 1000 ft.
Loss of Head per 1000 ft.
Loss of Head per 1000 ft.
1909
1910
1909
1910
1909
1910
Observations
Loss of Head
c
n
1909
1910
Velocity, in Feet per Second
Loss of Head per 1000 ft.

FIG. 14.

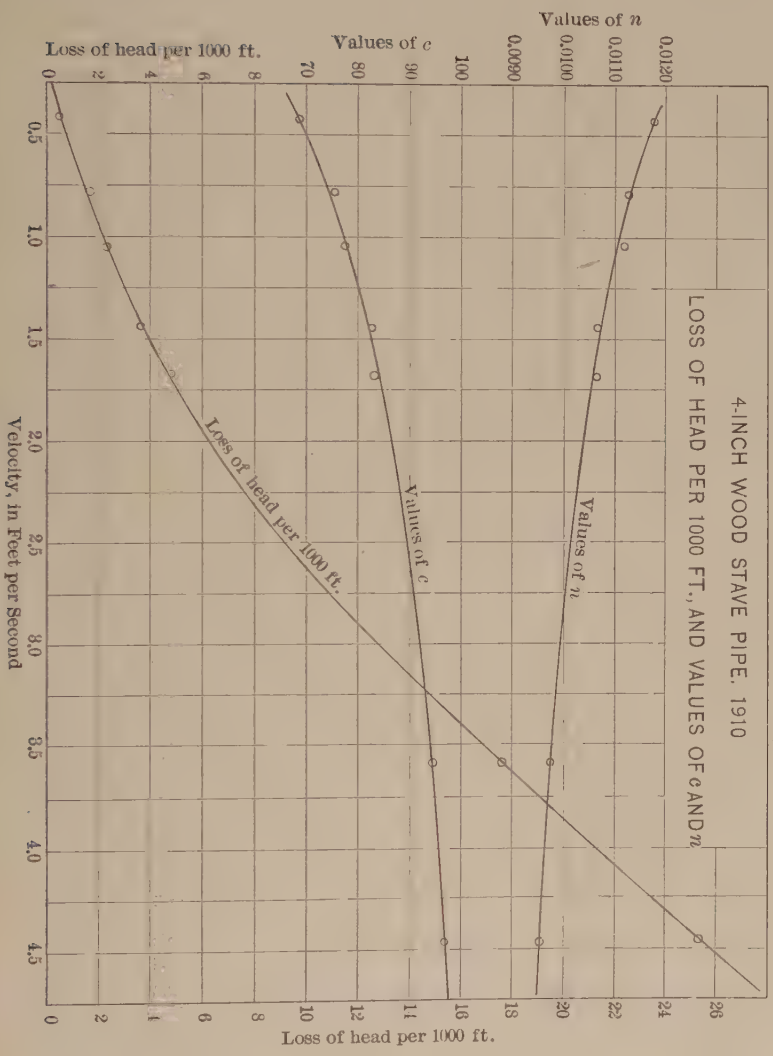

4-INCH WOOD STAVE PIPE. 1910
LOSS OF HEAD PER 1000 FT., AND VALUES OF c AND n
Values of n
0.0120
0.0110
0.0100
0.0090
Values of c
70
80
90
100
Loss of head per 1000 ft.
0
2
4
6
8
Values of n
Values of c
Loss of head per 1000 ft.
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
Velocity, in Feet per Second
0
2
4
6
8
10
12
14
16
18
20
22
24
26
Loss of head per 1000 ft.

Fig. 15.

conclusions in regard to any single pipe. This same argument would apply to the 4-in. pipe, also, in case the flow conditions here should be considered abnormal on account of the possible presence of fungous growth on the inner surface.

Proceeding now to the determination of the values of z and K: As already stated, the general equation of the dashed line of negative slope shown in the diagram is $m = K\ D^z$, the values of m being given on the scale on the top, and values of D being given at the right. In drawing this line the principal weight has been attached to the 5-, 6-, 8-, 14-, 18-, and 55¾-in. pipes. It happens that the point for the 22-in. pipe falls quite close to the average line, although no weight was given to it in determining this line. The reason for disregarding this point is that the slope is so widely at variance with the general average that it was not thought fair to give much more weight to the intercept point than was attached to the slope in the determination of the average value of x. The logic of this, however, cannot be demonstrated by the writer. It is a matter of opinion only. There is good reason for disregarding the point for the 12-in. pipe, as there is no doubt that the position of the 12-in. line is abnormal, and, as already stated, the presence of silt could account for the entire discrepancy in position, and, very probably, without altering the slope of the line. A similar argument can be applied to the 4-in. pipe, though not so conclusively. These arguments are based on hypotheses, not on known facts, and the writer is not prepared, at present, to defend them with facts. He believes, however, that the deductions are justified by the experiments until such time as evidence to the contrary is available.

Assuming now that the line as drawn represents the true average, its slope is calculated and gives $z = -1.263$; then, reading the intercept on the $D = 1$ line, $K = 8.6$.

All the constants in the general equation, $H = K\ D^z\ V^x$, have now been determined, and we may write

$$H = 8.6\ D^{-1.263}\ V^{1.8}$$

as the formula for loss of head in wood stave pipe, as determined by these experiments,

in which H = the loss of head, in feet per 1 000 ft.;

and D = the diameter of the pipe, in inches;

V = the mean velocity of flow, in feet per second.

The writer has adopted for his use the formula:

$$H = 8.6\ D^{-1.26}\ V^{1.8}$$

or, reducing this to a more practical form, and expressing the diameter in feet instead of inches:

$$Q = 1.353\ D^{2.70}\ H^{0.555}.$$

This formula is not recommended for adoption until more data are available and some of the uncertain points have been cleared up, but, until this is accomplished, the writer feels fairly safe in using it. With good conditions, and scientific and careful operation and maintenance, it should give results within 10% of the true quantities for velocities from 0.5 to 6 ft. per sec.

Values of Kutter's n as Applied to Wood Stave Pipes.

It has become generally recognized that Kutter's coefficients have no direct application to the flow of water in pipes, but, as additional proof of the fallacy of these coefficients, four good examples have been selected from the pipes covered by these experiments, and curves have been drawn representing the values of n and c (Chezy formula) and corresponding values of velocity. Incidentally, the diagrams also show the curves representing the relation of loss of head to velocity. For the 8- and 55¾-in. pipes two sets of such curves are shown, representing the results of the 1909 and 1910 experiments.

The curves selected show a comparatively uniform variation. The remaining experiments show a very non-uniform variation, and no simple curve can even approximate the locus of all the points. The values of n for the 22-in. pipe show a variation directly the opposite of that represented in the curves given, that is, the value of n increases with the velocity. However, in the light of the results for this pipe already given, this is not so surprising.

The writer has also prepared Table 11, showing the value of n, for each of the pipes tested, for velocities of 0.5, 1.0, 2.0, 3.0, and 4.0 ft. per sec. On account of the difficulty of obtaining an average line to represent all the points, many of the values are necessarily very rough. By comparing these values with diameters horizontally and with velocities vertically, it is evident at a glance that no uniform relations exist.

The writer knows of engineers who, in the absence of better data, have used a scale of values of n depending on the pipe diameter, using

smaller values for the smaller pipes. How this idea originated the writer does not know, but he does not believe that it is supported by any known facts. It is certainly not in harmony with the present experiments. There is just as much, and probably more, foundation in fact for making n vary with the velocity as with the diameter.

TABLE 11.—VALUES OF KUTTER'S n FOR VARIOUS DIAMETERS AND VELOCITIES.

Velocity.	Diameters, in Inches.												
	4	5	6 1909	6 1910	8 1909	8 1910	12	14 1909	14 1910	18	22	55¾ 1909	55¾ 1910
0.5	0.0117	0.0105	0.0103	0.0103	.....	0.0110	0.0130	0.0118	0.0104	0.0112	0.0096	.....	.....
1.0	0.0111	0.0107	0.0102	0.0105	0.0115	0.0103	0.0128	0.0114	0.0108	0.0109	0.0107	.....	0.0118
2.0	0.0104	0.0106	0.0100	0.0102	0.0111	0.0095	0.0122	0.0109	0.0108	0.0105	0.0120	0.0109	0.0113
3.0	0.0099	.....	.....	0.0100	0.0109	0.0091	.....	0.0105	0.0106	0.0103	0.0126	0.0105	0.0110
4.0	0.0096	.....	.....	.....	0.0107	0.0089	.....	.....	.....	.....	.....	.....	0.0108

COMPARISON WITH OTHER EXPERIMENTS.

A satisfactory comparison of these with other experiments previously published, notably those of Mr. T. A. Noble on 44- and 54-in. pipes, and Messrs. Marx, Wing, and Hoskins on a 72-in. pipe, is a difficult and discouraging problem. The writer had intended to make such a comparison and include it in this paper, but this would have taken more time than was available. Furthermore, the results of previous experiments are so at variance with those described in this paper that it is doubtful whether any satisfactory balance of all available data can be made. It is hoped, however, that some scientifically minded person will make a serious attempt to do this, and there is no doubt that he will receive the thanks of the Engineering Profession if he succeeds.

It is interesting to note the position of the points representing the values of m for the other tests with respect to the writer's average line. These points are indicated by solid black triangles. The only one that falls near the line is that for the 14-in. pipe tested by Adams, which falls almost exactly on it. For the 18-in. pipe by Adams, the 24-in. pipe by Gutelius, and the 30-in. pipe by Schuyler, only one test each was available, and the intercepts were determined by running through these points a line having a slope of 1.8. This is, of course, a make-

shift, and the resultant m points do not represent true experimental data. It happens that the observations on the 24-in. pipe were made at a velocity of about 1 ft. per sec., so that the m point could not be far off, whatever the slope of the line run through it. For the 44-, 54-, and 72-in. pipes, complete data are available, and the points shown are based on these.

It has been the intention, through this paper and in the various tables and diagrams, to present complete data, so that all reductions could be worked over in any form, and if this aim has not been realized, additional notes or information can be furnished to any one desiring them. The original field notes are on file in the office of the Reclamation Service, at Sunnyside, Wash.

The writer hopes that this paper contains valuable information, and that it will receive a thorough discussion, so that all doubtful and disputed points on this subject may be cleared up, as far as possible.

DISCUSSION

Mr. Dieck. Robert G. Dieck, M. Am. Soc. C. E. (by letter).—This paper is an important contribution to the subject of pipe gaugings. The great care which has evidently been taken in the author's investigations and the considerable range of the experiments, give peculiar emphasis to the results, while the direction of further experiments in this field is clearly indicated. The conditions surrounding the experiments were most favorable for close determinations of the friction factor, and the clear exposition of the case warrants careful and serious discussion.

The use of the Kutter formula in pipe design has always been questionable, even though its ease of application, in default of a more convenient formula, has commended it. The assumption of a value for n has always been an arbitrary one among hydraulic engineers; and on account of this indefiniteness, uniformity of results has never been possible. For wood stave pipe, values for this factor ranging from 0.009 to 0.012 have been assumed, but the information regarding it has been of such an insufficient nature that even a reasonable approximation to the truth has never been secured.

It is evident from the Sunnyside experiments that an adjustment in the ideas of hydraulicians on this point is bound to come, and that these experiments, even if found to be imperfect, will point the way to a more rational assumption of the factor, and perhaps be the means of discovering a simpler and more elastic formula. A close analysis of the experimental results will show that the choice of a value for n is not such a simple matter as has heretofore been assumed, and that, if full credence is to be placed in the results of Mr. Moritz's experiments, this factor is far from being a constant under all conditions of flow. When the discharge varies, all other conditions being the same, the value of n also varies; hence, in its present form, the Kutter formula cannot be considered a true statement of conditions. The introduction of a variable n would undoubtedly complicate the matter, and, in all probability, would make the application of the formula to pipe design practically an impossibility. However, as in all ordinary design, extreme accuracy is neither sought nor expected, further complication of the already involved Kutter formula, by the insertion of a value of n with a variable exponent, seems to be unnecessary. Analysis of the experiments will show that generally the factor decreases with increase in velocity, though all the results are not uniform in this respect, and though the variations do not follow a well-defined law.

It is to be expected that inaccurate determinations of the observed quantities have been made, and that certain contingencies might be met better if the experiments were retraced. The manner of connecting the manometers, the presence of air in the pipes, eddies in the

pipes, and other factors, have no doubt had effect on the work; therefore, a rigid comparison of results may be improper. From the paper it would seem that the method of computing the cross-sections of the pipes was not entirely proper, and certain anomalies might be removed if this feature of the work were again investigated. The author assumes that leakage has generally been negligible, but this assumption is open to doubt. The correction for temperature is generally so small that it may be neglected, and therefore its determination has only an academic value. Except where there are great differences between the temperature of the flowing water and that of the mercury columns, temperature corrections need not be applied. It might have been of peculiar interest, however, if record of the specific gravity of the flowing water had been made. Mr. Moritz assumes a constant reduction factor for the mercury of the gauges, but possibly there may be error in this. The author does not state that screens were inserted to remove floating matter, or that the pipes were blown off in advance of the experiments to remove sediment. It is suggested that floating matter in the neighborhood of the manometers may have affected the readings appreciably, and that deposits may have reduced the areas of the pipes under conditions of low velocity. Mr. Dieck.

Considering now the experimental results in detail, a general increase in n for the 55¾-in. pipe is noted in the series for 1910 over that for 1909. It might be expected that the interior surface of the pipe would become smoother with use. This might appear to indicate that deposits had reduced the area, but the gradient of nearly 3% in the section casts some doubt on this objection, though the velocity in the series has never exceeded 4 ft. per sec. The average value of n is 0.01072 for the 1909 series and 0.01111 for that of 1910. The curves of variation are practically parallel.

The lack of data on the 22-in. pipe makes it difficult to draw reasonable inferences from the experiments, and it seems wiser to exclude from discussion the results from that pipe. The curve of variation, which is of entirely different character from the others, would seem to bear out the assertion that there are serious leaks in the pipe, for, with increase of head, n increases considerably. The allowance for leakage has been small, and has not been varied with altered conditions of flow.

With the exception of Test No. 8, the curve of variation of n for the 18-in. pipe follows a general downward tendency with increased velocity. The average of the eight tests gives n a value of 0.01074. The profile of this pipe is such as to make reasonable the assertion that deposits had some influence on the results. With the exception of some irregularities, and a drop in value in Test No. 7, of the series for 1909, the curve of variation in n is regular, that is, inversely with the velocity. The average value is 0.01117 for the series. In the series for

Mr. Dieck.

1910, with the exception of that for Test No. 6, the value of n remains practically uniform for all velocities, the average of the determinations being 0.01075. It will be noted that the value has decreased with the use of the pipe.

With the 12-in. pipe, the value of n, while following the general rule of variation, is considerably higher than for all the other pipes, even excluding the apparently abnormal result obtained from Test No. 3. Omitting this test, the average value is 0.01276. The author's explanation—that there may have been deposits—seems sound.

Both series for the 8-in. pipe show the general tendency of variation in n, but the decided lowering of value in the series for 1910 is most interesting. Mr. Moritz objects to an explanation of this on the score of deposits, but rather favors the opinion that there was choking by air in the first series. It is possible, however, that in the early condition of the pipe the interior surface was quite rough, and engaged considerable sediment, thus decreasing the cross-section and apparently increasing the friction factor. With use and the occurrence of high velocities, the interior roughness in all probability did decrease, and his contention, therefore, is not adequate. The average values are 0.0111 for the 1909 series, and 0.00953 for that of 1910.

On the other hand, with the 6-in. pipe, there has been small change in the value of n throughout the entire series, though this factor shows an increase with use. Otherwise, there is no change in the general rule. The average values are 0.01022 for the 1909 series, and 0.01027 for that of 1910.

The curve of variation for the 5-in. pipe shows considerable irregularity. The meager information regarding the pipe makes strict discussion difficult. Here velocities are very low, and deposits may have occurred; but it will be noted that with the lowest velocity the friction factor is smallest; therefore, some other explanation of the peculiar results must be sought.

The average value of n is 0.01058. The general rule of variation is applicable to the tests on the 4-in. pipe, though the variation curve is very steep. The average value of n (0.01111 for Tests Nos. 3 to 7, inclusive) would not lead to the belief that there was serious clogging from the growth noted. The small values obtained from Tests Nos. 1 and 2, however, may be taken as an indication that there was some deposit under low velocities, due in part to the presence of the growth.

Taking into consideration the entire group of experiments, it is reasonable to conclude:

1. That the value n for wood stave pipe, whether continuous or jointed, increases with decrease in velocity (or varies inversely with some power of the velocity), and that, for practical purposes, this value lies within the rather narrow limits of 0.0100 and 0.0110;

2. That greater regularity in the determinations is evident when the loss of head has been measured by mercury gauges; Mr. Dieck.
3. That the assumption of an invariable reduction factor for the mercury of the gauges is not altogether proper;
4. That methods to reduce the quantity of air in the pipes and the quantity of silt entering do not seem to have been adopted;
5. That there is no great necessity for altering the Kutter formula.

Undoubtedly, the experiments should be continued, for a number of doubtful points would then be more clearly understood.

MELVIN L. ENGER, ESQ.* (by letter).—The following statements are supported by reliable experiments on the loss of head in pipes, when the critical velocity is exceeded: Mr. Enger.

1. The exponent, x, in the equation, $H = m V^x$ is an index of the smoothness of the interior surface of the pipe.
2. The value of x may be as low as 1.70 for very smooth, clean pipe, and as high as 2.00 for pipes with a very rough interior surface.
3. Under no condition will the value of x exceed 2.00.

If the last statement is correct, some error must have been made in the experiments with the 22-in. pipe. The writer has plotted to logarithmic scale, in Fig. 16, the results given in Table 3 for the 22-in. pipe. The values from this table are represented by solid circles, and these do not lie on a straight line. Adding 0.14 ft. to each value of the lost head puts five of the seven points on a straight line. Adding an amount larger than 0.14 makes the curve passing through these points concave upward; a smaller value makes the curve concave downward. The points obtained by adding 0.14 ft. are represented by open circles.

The amount (0.14 ft.) which it seems should be added to the loss of head may be explained as due to an error of that amount in the difference in the elevations of the zeros of the piezometers, or it may be due to an accumulation of air at a summit in the pipe line, or in one of the gauges.

When the correction is made, the equation for the 22-in. pipe becomes $H = 0.222 V^{1.95}$, instead of $H = 0.161 V^{2.15}$, as derived by the author.

J. L. CAMPBELL, M. AM. SOC. C. E. (by letter).—In 1907 and 1908, the writer constructed, as a part of a pipe line system 135 miles long, 12 700 ft. of 10-in. and 48 000 ft. of 16-in. wood stave pipe, in the order named, from Bonito Creek to Nogal Reservoir, New Mexico.† Mr. Campbell.

At the junction of the two pipes, an open stand-pipe, 6 in. in

* Asst. Professor of Theoretical and Applied Mechanics, Univ. of Illinois.

† *Transactions*, Am. Soc. C. E., Vol. LXX, p. 164.

Mr. Enger.

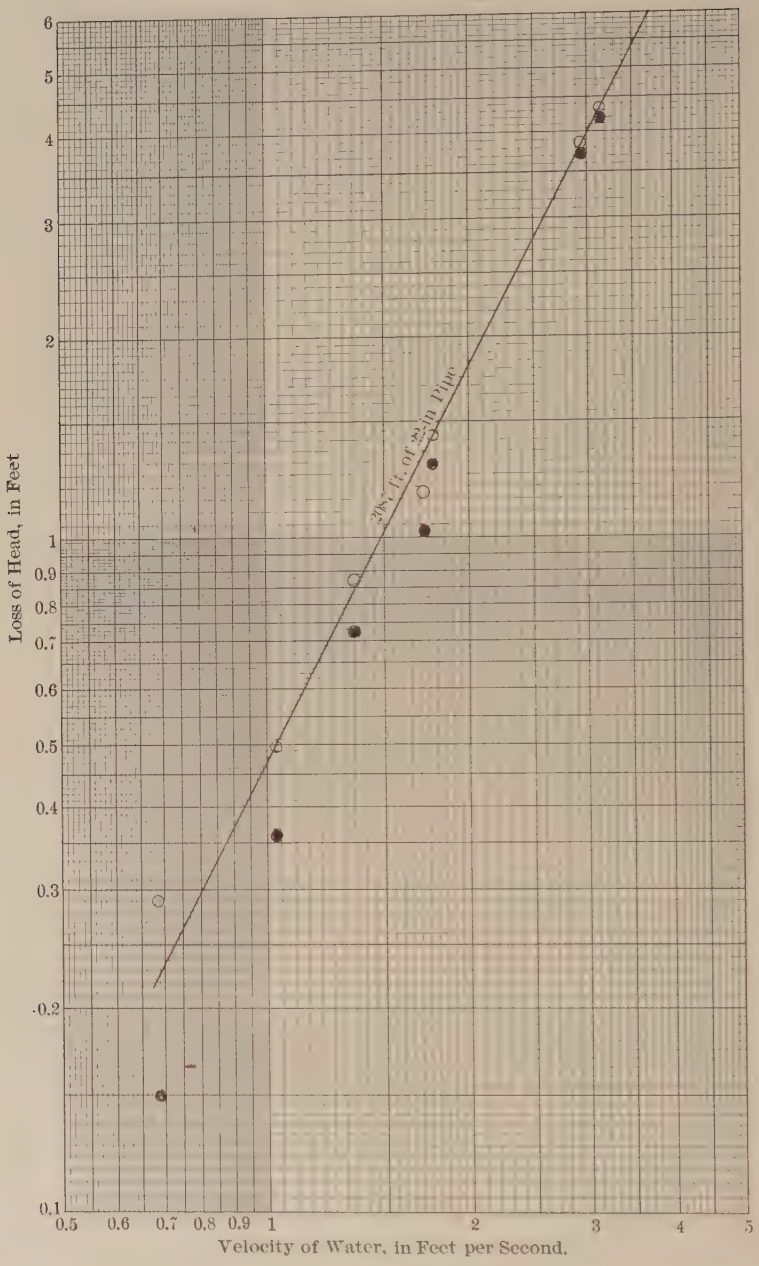


FIG. 16.

Mr. Campbell.

diameter, rises above the hydraulic grade line. The water surface elevations with maximum flow are as follows:

On the head of the 10-in. pipe	7 729.00
In the stand-pipe	7 305.00
In the outlet box at the reservoir	7 158.00

This gives gradients as follows:

For the 10-in. pipe	33.38+ per 1 000
" " 16-in. "	3.0625 " "

This pipe is like the jointed pipe described by the author, except that it is made of white pine, which takes a smoother surface in the planing mill than Oregon or Douglas fir.

The line was put in service in February, 1908. On March 10th and 31st, 1908, and in May, 1909, velocity measurements of flow were made. When the first two were made, there were 500 ft. of 10-in. pipe temporarily at the lower end of the 16-in. line, but this was replaced by 16-in. before the third measurement occurred.

In the first test, bran only was used. It flowed through the line from the head of the 10-in. pipe to the mouth of the 16-in. pipe, a distance of 60 700 ft., measured on the surface of the ground, in 3 hours 50 min., from which, the velocity and discharge were as follows:

Size of pipe.	Velocity.	Discharge.
10-in.	9.74 ft. per sec.	5.31 cu. ft. per sec.
16-in.	3.81 " " "	5.32 " " " "

In the second test, two colors and bran were used at 15-min. intervals, and with an observer at a test valve at the stand-pipe in addition to the men at the head and the mouth of the line. The mean time, velocity and discharge were:

Size of pipe.	Time.	Velocity.	Discharge.
10-in.	0 hours 21.83 min.	9.70 ft. per sec.	5.29 cu. ft. per sec.
16-in.	3 " 29.17 "	3.80 " " "	5.30 " " " "
Entire line	3 " 51. "		

In the third test, 15 months later, green aniline only was used three times at 30-min. intervals, with the following result:

Size of pipe.	Time.	Velocity.	Discharge.
10-in.	.....	10.71 ft. per sec.	5.84 cu. ft. per sec.
16-in.	.....	4.18 " " "	5.84 " " " "
Entire line	3 hours 31 min.		

The discharges were estimated on the nominal diameters of the pipe. Subsequently, measurements showed that the actual diameters are 10⅛ in. and 16⅛ in., respectively, from which the corrected discharges are:

Size of pipe.	Test.	Discharge.
10-in.....	No. 1	5.44 cu. ft. per sec.
	No. 2	5.42 " " " "
	No. 3	5.98 " " " "

Mr. Campbell.

Size of pipe.	Test.	Discharge.
16-in.	No. 1	5.40 cu. ft. per sec.
	No. 2	5.39 " " " "
	No. 3	5.93 " " " "

Before the third test was made, a standard weir, 4 ft. long, without end contractions, was built at the outlet box, and measured the flow from the 16-in. line, giving a discharge of 5.88 cu. ft. per sec., as compared with 5.93 cu. ft. per sec., estimated from the measured velocity—a discrepancy of about 1 per cent.

For the following season of 1910 the highest recorded discharge over the weir was 5.88 cu. ft. per sec., but, for eleven consecutive days in May, 1911, this weir recorded a flow of 6.19 cu. ft. per sec., from which the maximum velocities appear to be:

Size of pipe.	Velocity.
10-in.	11.07 ft. per sec.
16-in.	4.37 " " "

With these velocities in Kutter's formula, n and c become:

Size of pipe.	n	c
10-in.	0.0089	132.
16-in.	0.0094	137.

This appears to indicate that the velocity may have some effect on the value of n.

Mr. Hering.

Rudolph Hering, M. Am. Soc. C. E. (by letter).—The author practically opens his paper with the apparently disheartening remarks that the tests quoted by him, and the best available regarding the flow in wooden pipes, "are replete with inconsistencies." Later, he states: "any attempt to co-ordinate the results is apt to be discouraging" and "the results of previous experiments are so at variance with those described in this paper that it is doubtful whether any satisfactory balance of all available data can be made."

These opinions have been expressed rather frequently when new gaugings have been published, and it would almost seem as if engineers might be in error, either in some of their methods or in their expectations from them.

The published results of the gaugings quoted by the author, including his own, indicate that such gaugings were neither carelessly made nor could they be erroneous. In fact, they have usefully served the Profession in advising works which have been satisfactory.

Different results and varying conclusions are sometimes obtained simply because the details of the measurements as well as the conditions are not the same. Therefore one should hesitate to condemn the results unless one can compare very thoroughly all the details and justly pass upon their relative merits.

It has also seemed to the writer that the reason for some of these apparent inconsistencies was due to the manner in which the results

were expressed; in other words, due to the particular formula used. The cause of all the trouble probably does not lie in the facts themselves, but in their interpretation. This question will be reverted to later. Mr. Hering.

The common criticism, that the so-called Kutter formula is too complicated, is not a good objection. The formula is hardly ever used in its original form, but by tables and diagrams. For small sections, the cause of the complication, namely, the influence of the slope on the value of the coefficient, c, in the fundamental Chezy formula, is entirely negligible, which reduces the Kutter formula to a very simple form, in fact, simpler than a number of other formulas.

Nor has the condemnation of the Kutter formula for the last thirty years prevented its extensive use, or even prevented the author from giving it a prominent place in his investigations. Comparing the results given by him in Table 11, for pipes from 4 to 55¾ in. in diameter, it is almost astonishing to observe their comparative uniformity and consistency, when it is realized that this formula was neither devised from nor intended for such small pipes, or even for pipes at all, but only for open canals and rivers; and was based on gaugings on streams ranging in size from mill races up to the Mississippi River. Could such relative consistency have been possible if the framework of the formula had been very wrong?

The author says: "It has become generally recognized that Kutter's coefficients have no direct application to the flow of water in pipes," and, as additional proof of the asserted fallacy of these coefficients, he selects four curves from his experiments, to show that n is a variable.

In 1888, when the writer translated Kutter's work, in conjunction with John C. Trautwine, Jr., Assoc. Am. Soc. C. E., he recognized, as well as did Mr. Kutter himself, almost at the outset, that n was not to be considered a precise and unvarying constant, although it was more nearly so than any other constant before proposed. It was for this reason that the writer published in the translation, not Kutter's own tables with averages for n, but entirely new tables, in which the variations of n and not constant averages were shown, together with the other elements, as v, r, and s, so that the character and magnitude of this variation of n could be seen more plainly for all the gaugings available at that time. The coefficients given in the large table of the translation, therefore, are not fallacies, in the faintest degree. Every one was determined from actual gaugings, and appears merely in the dress—so to speak—of the Kutter formula, because the book happened to treat of that formula. Similar numerical values of coefficients representing the frictional and other resistances could be computed from the same gaugings in the dress of other formulas, such as Bazin's, Siedeck's, and others.

Mr. Hering.

The only fallacy contained in the Kutter formula is the one that caused its complicated form. It was made necessary by the published formula of Humphreys and Abbot, which was supposed to represent the gaugings of the flow in the Mississippi River, which, later, was unfortunately found to be erroneous in some of the assumptions. Leaving out the complication caused by this fact, the Kutter formula, as already stated, becomes much simplified. Bazin's new formula, by omitting the deductions from the erroneous Humphreys and Abbot formula, is better than Kutter's.

All these formulas, however, are mere forms, and give identically the same practical results within the range of the gaugings if to them are applied coefficients which are based on these actual gaugings. If there was a table of coefficients for Bazin's and others for Siedeck's formula, for each case of size, shape, slope, and degree of roughness of wetted perimeter, then, if they were derived from actual gaugings within the same range, the same results would be obtained as with the Kutter formula with the Kutter coefficients.

The mean velocity of flow is caused by the hydraulic grade plane, and is controlled by all the resistances to the flow, one being the mean depth of water, and another the friction against the perimeter.

The latter cannot be exactly determined so as to be expressed by any formula. It can at best only be approximated and averaged. Differences in the results of different formulas are often due to the approximations alone. The exact determination of the resulting mean velocity of flow is only possible by an accurate gauging of the case in question, and, for the practical application of any formula to a new case, the engineer should always consult actual gaugings, approximating as nearly as possible to the new case he has on hand.

The mean depth of water is expressed usually as "mean depth" for rivers and as "mean radius" for artificial channels. There is a difference, sometimes of moment, between these expressions. The applications of formulas, therefore, should take due account of it.

Take three artificial channels of the same sectional water area, and let one be circular and two rectangular, of which one is deep and the other shallow. The two different rectangular sections have identically the same "mean radius," but the deeper one has 50% greater "mean depth." The circular section of the same cross-sectional area has almost one-fourth less "mean radius" than either, and a "mean depth" 50% greater than that of the shallow rectangle.

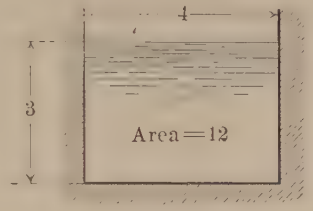
All the older mean-velocity formulas take the mean radius as a constant, as well as the slope, and put all the variables into the coefficient, c, in the fundamental formula $v = c \sqrt{r s}$. This awkward feature, of itself, may bring trouble into the results, when comparing different cross-sections. The new formula of Siedeck, by introducing

the shape as well as the size of the cross-section, obviates this feature, but it is relatively complicated, and professes to be applicable only to open channels. Mr. Hering.

On the other hand, in defence of the older practice, it must still be insisted that the friction of water against a firm surface, contrary to the friction of one solid against another, varies with area and not with pressure. Therefore, the area of friction, that is, the wetted perimeter, cannot be entirely ignored, which then makes the mean radius a better measure than the mean depth.

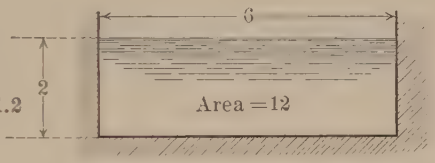
$$\text{Mean Depth} = \frac{\text{Area}}{\text{Width}} = \frac{12}{4} = 3.0$$

$$\text{Mean Radius} = \frac{\text{Area}}{\text{Wet Perim.}} = \frac{12}{10} = 1.2$$



$$\text{Mean Depth} = \frac{\text{Area}}{\text{Width}} = \frac{12}{6} = 2.0$$

$$\text{Mean Radius} = \frac{\text{Area}}{\text{Wet Perim.}} = \frac{12}{10} = 1.2$$



$$\text{Mean Depth} = \frac{\text{Area}}{\text{Width}} = \frac{12.0}{3.91} = 3.07$$

$$\text{Mean Radius} = \frac{\text{Area}}{\text{Wet Perim.}} = \frac{12.}{12.3} = 0.98$$

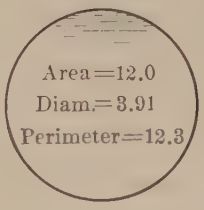


FIG. 17.

All these troubles and discouragements, as expressed by the author, from doubts, inconsistencies, apparent errors, and discrepancies in the formulas for the mean velocity of water, can be obviated with the use of almost any one of the best formulas by simply using it together with a table of experience data. Such a table should have an orderly classified enumeration of the results of actual and carefully made gaugings, so that, instead of by computations, the true relation can be obtained from it by inspection, and based on the following factors to

Mr. Hering.

be given in the table: velocity, cross-sectional area, surface width of channel, mean depth, mean radius, slope, and a numerical value for a coefficient representing the resistances computed from whatever formula may be the basis of the table, and giving as full a description as practicable of the character of the surface on which the water flows, of the horizontal configuration and length of the gauged channel, and mentioning any other causes which might have produced a velocity retardation or acceleration in the specific gaugings.

As gaugings covering almost every ordinary case are now available, a table such as suggested would dissipate the necessity for any formula, except to interpolate safely the conditions between those of known gaugings. For ordinary use, a few average coefficients, such as those now generally given, would suffice. For important cases, a sufficiently wide range of actual gaugings, all reduced to one and the same formula, should be used.

The writer, following these thoughts, made an attempt to construct such a table in 1888 in the translation of the book by Ganguillet and Kutter. The results in this table range from those obtained with small pipes to those of the Mississippi River, the table recording more than 1 250 actual gaugings. Since the date of publication, some of these gaugings have been found untrustworthy because carelessly or wrongly made, but a large number of new and excellent gaugings have also been made, among which may be mentioned those of the author and those quoted by him, to say nothing of a multitude of stream gaugings. If these were all added to such a table, including descriptions of the kind mentioned, it seems that the hydraulic engineer should be supplied with sufficient practical information from actual gaugings to interpolate safely the desired coefficients for almost any special cases he might have on hand. All new gaugings, such as the author's, would find their place in such a table, and add to the usefulness of the whole.

This proceeding, by using a formula merely as a frame for experience data, would be far safer than to use any formula alone for a new case, particularly when it has been obtained from a limited set of gaugings, unless the new case is an almost identical one.

Exponential formulas are readily obtained from any set of experiments, and the author has given one from his experiments. Yet it must not be forgotten that their range of application is limited to their specific source, and they rarely apply to other conditions. For this reason they are rarely heard of again. The *Transactions* of this Society contain a number of them which apply to the flow of water in pipes. They fitted their special cases admirably, as does the author's formula, but their usefulness and advantage generally end therewith.

Mr. Williams.

GARDNER S. WILLIAMS, M. AM. SOC. C. E. (by letter).—The experiments presented by the author have in them elements of so much value as to warrant a request for some additional data. In the paper, the information regarding the measurement of the loss of head is quite complete, but this is only half the problem. For the determination of the volume of flow, one is compelled to accept the author's bare statements, as the data presented are quite deficient in those elements essential to an independent computation. The author's notion of determining the velocity of approach with a current meter does not seem to indicate a great familiarity with weir measurements, and there are many who do not consider the current meter sufficiently reliable for such investigations as these. On that account the writer would ask that the author supply, if possible, the following additional information:

1. A description of the head gauge and its location, and of the method of reading the head on the weir, as well as of determining the gauge reading which corresponds to the elevation of the weir crest;

2. The area and dimensions of the channel of approach, (*a*), at the transverse plane through the head gauge, and (*b*), at the weir;

3. The exact dimensions of the weir notch, the length of the end contractions, and the height of the weir crest above the bottom of the channel of approach;

4. A plan showing the arrangement and location of the weir and the gauge, and the channel of approach.

It is rather surprising to find a hydraulic formula presented before this Society, in which one essential dimension is given in inches while the other dimensions are in feet. Our mechanical engineering friends are accustomed to jumbling up feet and inches in their formulas, but the civil engineer usually prides himself on sticking to a single unit wherever possible, and as all other pipe formulas in common use are in units of feet, it is to be regretted that the author has not put his in the same terms, to facilitate comparison, if for no other reason.

With the above information added, this paper is likely to become one of the very valuable contributions of recent years, as it covers a wide range of experiments in a hitherto unexplored field, and the writer would express his appreciation of the evidence of painstaking work on the part of the investigator, notwithstanding the foregoing criticism.

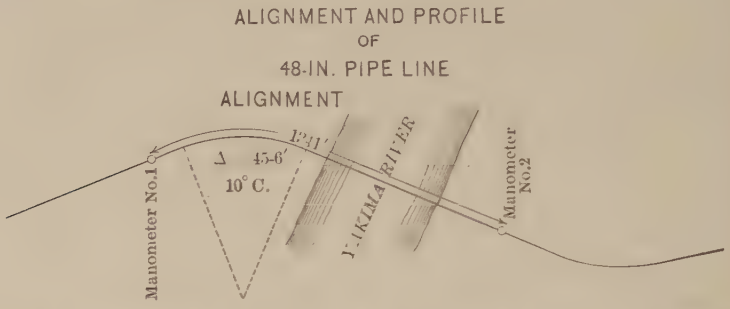
Mr. Moore.

J. S. MOORE, ESQ.* (by letter).—As additional data on the flow of water in wood pipe, the writer submits the results of two experiments on the Sunnyside Project, during the irrigation season of 1911, on 48¾- and 31-in. wood pipes of continuous stave construction.

* Junior Engineer, U. S. Reclamation Service.

Mr. Moore.

48¾-Inch Pipe.—The 48¾-in. pipe comprises that portion of the Mabton pressure pipe which lies under and adjacent to the Yakima River. Fig. 8 shows its location with respect to the 55¾-in. pipe, and Fig. 18 presents the alignment and profile. The maximum length available was 1 341 ft.



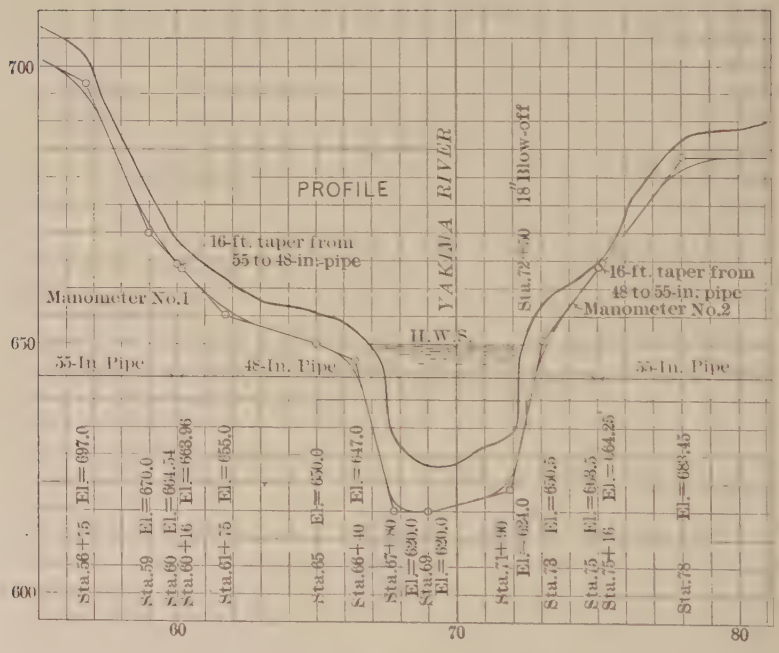


Fig. 18.

The pipe had been in operation for two and one-half irrigation seasons, during which time the maximum discharge had been about 70 cu. ft. per sec., thus producing a velocity of about 5.4 ft. per sec. At the time the observations were made, the requirements of the irrigation

Mr. Moore.

TABLE 12.—Observed and Calculated Data on 48¾-Inch Continuous Wood Stave Pipe.

No.	Date, 1911.	Head on 18-ft. weir.	Discharge from rating table, c. f. s.	Mercury Gauge No. 1. $C = 7.179$ ft. Elev. of "L" zero = 95.089.					Mercury Gauge No. 2. $C = 7.070$ ft. Elev. of "L" zero = 99.341.					Loss of Head.		Area.	Velocity.	n.
				U.	L.	Mercury.	Water equivalent.	Elevation of water column.	U.	L.	Mercury.	Water equivalent.	Elevation of water column.	Length, 1 341 ft.	Loss per 1 000 ft.			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)
1	July 20	0.588	30.0	0.8488	0.023	8.0048	108.248	206.287	0.8820	0.0755	7.8765	106.566	205.907	0.380	0.283	12.96	2.315	0.01113
2	July 20	5.667	36.4	0.9070	0.023	8.0630	109.035	207.074	0.9346	0.0750	7.9296	107.283	206.624	0.450	0.335	12.96	2.809	0.01013
3	July 21	0.779	46.2	1.0107	0.023	8.1667	110.437	208.476	1.0112	0.0750	8.0062	108.319	207.660	0.816	0.608	12.96	3.565	0.01078

The temperatures of the mercury columns were very nearly identical, and in the neighborhood of 35° cent. The density of the mercury at this temperature is taken as 13.52. As the mercury columns were practically equal in length, no correction has been applied for variation in water temperature.

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system would not permit the discharge to be reduced to less than 30 cu. ft. per sec., or to be increased to more than 48 cu. ft. per sec. The method of obtaining the discharge, from the heads over the 18-ft. rectangular weir at the intake, has already been described, in the paper, in connection with the 55¾-in. pipe. Several measurements of diameter were made, at irregular intervals, on the upper side of the river, at the time those on the 55¾-in. pipe were obtained, and were found to be very uniform. From these measurements the average area was calculated to be 12.96 sq. ft. Mercury columns were used at both pressure stations. No correction for temperature variations was made, because the columns were practically equal in length, and the thermometers indicated only slight differences in temperature. The observed and calculated results are given in Table 12.

It is to be regretted that a wider range of velocities was not secured, or that a greater number of observations, within the prescribed limits, was not made, so that more weight could be attached to the results. The writer found, however, that the maximum time allowable for the limited variation in the discharge, from the normal flow through the pipe, was entirely insufficient for a thorough investigation of the intermediate values.

The discrepancy in Observation 2 can be explained only by assuming a mistake, on the part of one of the pressure gauge observers, either in recording an erroneous reading of the mercury height, or in closing down the valve between the instrument and the main pipe to such an extent that the pressure was reduced, but not enough to lower the column any great amount. An inspection of the maximum and minimum records, taken at Manometer No. 1 for this observation, points strongly to this conclusion.

31-Inch Pipe.—The 31-in. pipe was completed in the spring of 1911, and water was first turned into it for operation in May, 1911. Just prior to the beginning of back-filling, and following the reeinching of bands and testing for leaks, measurements of the external diameters were taken at intervals of 50 ft. throughout the entire length. Consequently, it was impracticable to obtain vertical diameters in this manner, so that only two diameters, at angles of 45°, were measured at each point. The thickness of a number of staves had been secured, and the average of these figures was used in determining the internal diameters, a slight correction being made for the swelling of the staves. The average area for the entire length was thus found to be 5.252 sq. ft., so that no great error is introduced in using the area of 5.241 sq. ft., derived from a diameter of 31 in. The variation of area is given in Fig. 19, which shows also the location of the manometers. A slight reduction in area is observed just ahead of Manometer No. 2, but this decrease is not sufficient to cause any perceptible difference in the pressure at that station.

Utilizing his experience in determining the discharge over the 18-ft. weir at the intake to the 55¾-in. pipe, the writer was prompt to make use of the current meter as a check on the weir at the intake to this pipe. These measurements were made from a gauging bridge, located a short distance above a 6-ft. Cippoletti weir, in a manner similar to that described for the 55¾-in. pipe. The resulting average discharge curve and rating table derived therefrom are shown by Fig. 20 and Table 13, respectively. In Fig. 20 is also shown, for comparison, the curve of discharge obtained by the direct application of the Cippoletti

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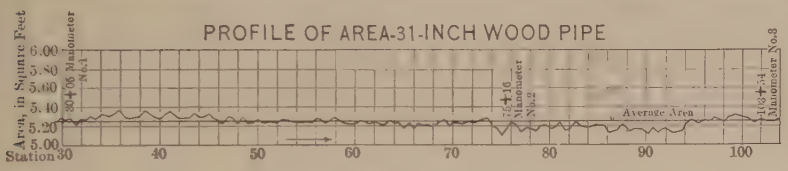

FIG. 19.

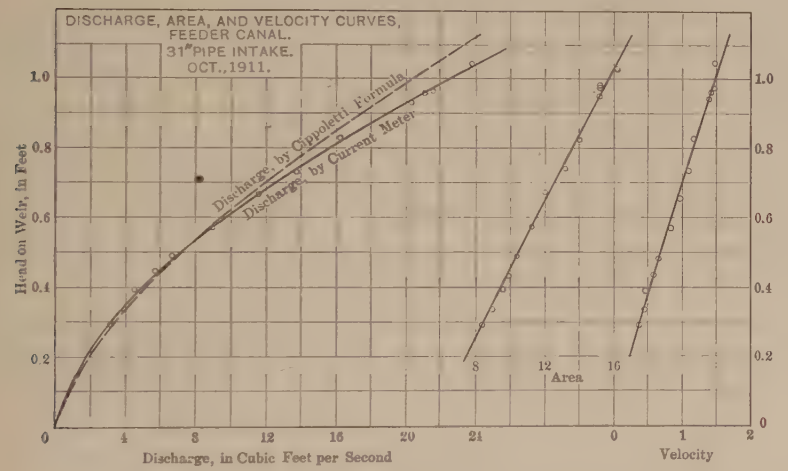

FIG. 20.

formula, without correction for velocity of approach or for incomplete contraction. The agreement between these curves for heads less than 0.6 ft. over the weir is all that could be desired, while above this limit the divergence results from the increase in velocity of approach, combined with the effect of incomplete contraction.

The alignment and profile are presented in Fig. 21, and Table 14 gives the results of the observations. In view of the fact that the maximum discharge would not be allowed to pass through the entire length of the pipe, at a time when the maximum variation in the flow could be made, measurements of the loss for this maximum quantity

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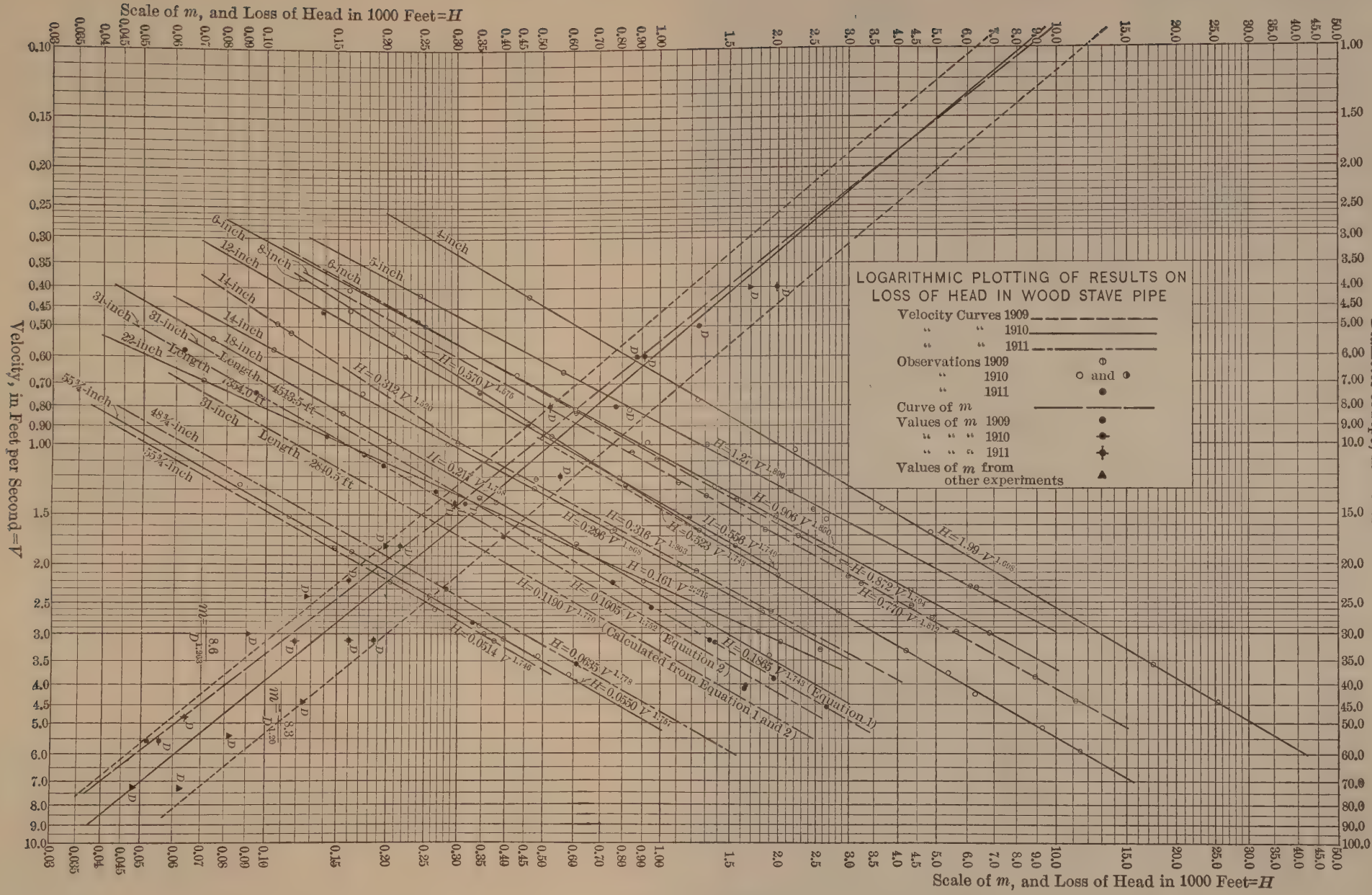
were obtained during August. Observations were taken on the total available length of 7 354 ft., and also, as a check, on the 4 513.5 ft. contained between Manometers Nos. 1 and 2. The agreement between these results for the different lengths showed that the friction losses were uniformly distributed throughout.

TABLE 13.—RATING TABLE FOR 6-FOOT CIPPOLETTI WEIR IN FEEDER CANAL AT 31-INCH PIPE INTAKE, FROM AUGUST TO OCTOBER, 1911.

Gauge height, in feet.	Discharge, in second-feet.	Difference, in second-feet.	Gauge height, in feet.	Discharge, in second-feet.	Difference, in second-feet.	Gauge height, in feet.	Discharge, in second-feet.	Difference, in second-feet.	Gauge height, in feet.	Discharge, in second-feet.	Difference, in second-feet.	Gauge height, in feet.	Discharge, in second-feet.	Difference, in second-feet.
0.200	1.68		0.400	5.00		0.600	9.60		0.800	15.51		1.000	22.33	
		0.13			0.20			0.26			0.32			0.36
0.210	1.81		0.410	5.20		0.610	9.86		0.810	15.83		1.010	22.69	
		0.13			0.20			0.27			0.32			0.36
0.220	1.94		0.420	5.40		0.620	10.13		0.820	16.15		1.020	23.05	
		0.14			0.21			0.27			0.33			0.37
0.230	2.08		0.430	5.61		0.630	10.40		0.830	16.48		1.030	23.42	
		0.14			0.21			0.28			0.33			0.37
0.240	2.22		0.440	5.82		0.640	10.68		0.840	16.81		1.040	23.79	
		0.15			0.21			0.28			0.33			0.37
0.250	2.37		0.450	6.03		0.650	10.96		0.850	17.14		1.050	24.16	
		0.15			0.22			0.28			0.33			0.37
0.260	2.52		0.460	6.25		0.660	11.24		0.860	17.47		1.060	24.53	
		0.16			0.22			0.29			0.33			0.37
0.270	2.68		0.470	6.47		0.670	11.53		0.870	17.80		1.070	24.90	
		0.16			0.22			0.29			0.34			0.38
0.280	2.84		0.480	6.69		0.680	11.82		0.880	18.14		1.080	25.28	
		0.16			0.23			0.29			0.34			0.38
0.290	3.00		0.490	6.92		0.690	12.11		0.890	18.48		1.090	25.66	
		0.17			0.23			0.30			0.34			0.38
0.300	3.17		0.500	7.15		0.700	12.41		0.900	18.82		1.100	26.04	
		0.17			0.23			0.30			0.34			
0.310	3.34		0.510	7.38		0.710	12.71		0.910	19.16		.....	.....	....
		0.17			0.23			0.30			0.34			
0.320	3.51		0.520	7.61		0.720	13.01		0.920	19.50		.....	.....	....
		0.18			0.24			0.30			0.35			
0.330	3.69		0.530	7.85		0.730	13.31		0.930	19.85		.....	.....	....
		0.18			0.24			0.31			0.35			
0.340	3.87		0.540	8.09		0.740	13.62		0.940	20.20		.....	.....	....
		0.18			0.24			0.31			0.35			
0.350	4.05		0.550	8.33		0.750	13.93		0.950	20.55		.....	.....	....
		0.18			0.25			0.31			0.35			
0.360	4.23		0.560	8.58		0.760	14.24		0.960	20.90		.....	.....	....
		0.19			0.25			0.31			0.35			
0.370	4.42		0.570	8.83		0.770	14.55		0.970	21.25		.....	.....	....
		0.19			0.25			0.32			0.36			
0.380	4.61		0.580	9.18		0.780	14.87		0.980	21.61		.....	.....	....
		0.19			0.26			0.32			0.36			
0.390	4.80		0.590	9.34		0.790	15.19		0.990	21.97		.....	.....	....
		0.20			0.26			0.32			0.36			

This table is not applicable for ice or obstructed channel conditions. It is based on thirteen discharge measurements made during August and October, and is well defined between gauge heights, 0.8 ft. and 1.0 ft.

Toward the close of the season the experiments were resumed, but nothing in excess of 9 cu. ft. per sec. could be allowed to pass through the entire length. Any discharge exceeding this was required to be wasted into the river through the waste-valve, the location of which is shown in Fig. 21. Observations were taken first on the 4 513.5-ft.


Scale of m, and Loss of Head in 1000 Feet=H
Velocity, in Feet per Second=V
Diameter of Pipe, in Inches=D
Scale of m, and Loss of Head in 1000 Feet=H
LOGARITHMIC PLOTTING OF RESULTS ON LOSS OF HEAD IN WOOD STAVE PIPE
Velocity Curves 1909
" " 1910
" " 1911
Observations 1909
" 1910
" 1911
and
Curve of m
Values of m 1909
" " " 1910
" " " 1911
Values of m from other experiments
4-inch
5-inch
6-inch
8-inch
12-inch
14-inch
18-inch
22-inch
31-inch
48¾-inch
55¾-inch
Length 354.0 ft.
Length 451.5 ft.
Length 2840.5 ft.
H=0.570 V1.876
H=0.312 V1.920
H=0.214 V1.759
H=1.27 V1.896
H=0.906 V1.850
H=0.558 V1.746
H=0.523 V1.743
H=0.316 V1.869
H=0.296 V1.869
H=0.161 V2.015
H=1.99 V1.668
H=0.852 V1.794
H=0.740 V1.819
H=0.1190 V1.750
H=0.1605 V1.752
H=0.1865 V1.743 (Equation 1)
(Calculated from Equation 1 and 2)
(Equation 2)
H=0.0514 V1.746
H=0.0635 V1.778
H=0.0550 V1.755
m=8.6/D1.903
m=5.3/D1.20

length, until the flow was reduced to the limit of 9 cu. ft. per sec., after which the waste-valve was closed, the instrument was removed to the outlet pressure station, and the observations were resumed over the total length. The agreement between the losses in these different lengths, as obtained in August, led the writer to believe that the same agreement would result for the later experiments, otherwise he would certainly have maintained Manometer No. 2 for the small velocities. The grave disparity in the two results is shown on the logarithmic

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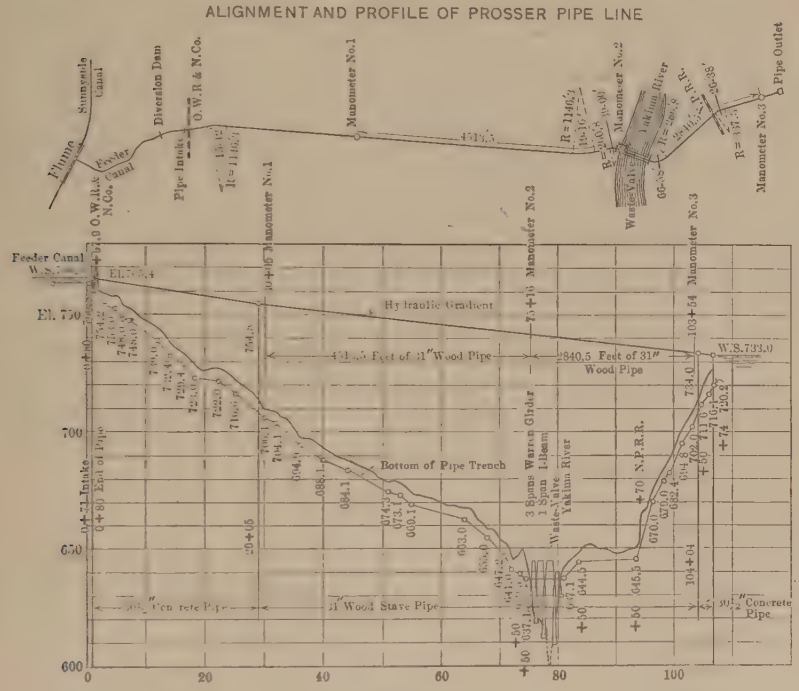

FIG. 21.

plotting on Plate LIX. The two curves, derived from six observations, are distinctly separate and well defined. The difference in the exponents of velocity is not large, but the writer is not prepared to explain the difference in the value of m. It has the appearance of a local loss at some point or points in the 4 513.5-ft. length, which was evidently formed between August and October.

Working on the assumption of some abnormal loss in this length, the writer has calculated from the two equations an equation to represent the loss in the remaining length of 2 840.5 ft. This curve is also shown on Plate LIX, and it is doubtful if the results for the

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TABLE 14.—Observed and Calculated Data

No.	Date, 1911.	Head on 6-ft. weir.	Discharge from rating table, c. f. s.	Mercury Gauge No. 1.										C.
				C.	U.	L.	Mercury.	T° cent.	t° cent.	Mercury reduc.	Water equivalent.	Elev. of "L" zero.	Elev. of water column.	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
1	Aug. 7	0.972	21.32	2.012	0.505	0.028	2.489	27.1	22.2	2.486	33.788	710.986	744.774	1.170
2	Aug. 7	0.966	21.11	2.012	0.497	0.028	2.481	21.6	21.6	2.481	33.720	710.986	744.706	7.199
3	Oct. 16	0.944	20.34	2.029	0.310	0.090	2.249	19.5	13 3	2.247	30.604	711.017	741.621	6.511
4	Oct. 16	1.047	24.05	2.029	0.632	0.090	2.571	14.2	12.8	2.571	35.004	711.017	746.021	6.511
5	Oct. 17	0.831	16.51	1.023	1.091	0.093	2.021	12.8	11.1	2.021	27.538	711.013	738.551	6.511
6	Oct. 17	0.827	16.38	1.023	1.105	0.092	2.036	14.5	11.7	2.035	27.727	711.013	738.740	6.511
7	Oct. 17	0.738	13.56	1.023	0.959	0.092	1.881	16.4	12.2	1.880	25.622	711.013	736.635	6.511
8	Oct. 17	0.672	11.59	1.023	0.878	0.092	1.809	17.8	12.8	1.807	24.631	711.013	735.644	6.511
9	Oct. 17	0.573	8.90	1.023	0.794	0.092	1.725	15.5	12.8	1.724	23 503	711.013	734.516	1.149
10	Oct. 17	0.488	6.87	1.023	0.707	0.093	1.637	15.0	12.8	1.336	22 310	711.013	733.323	1.149
11	Oct. 17	0.444	5.90	1.023	0.665	0.093	1.595	10.8	12.8	1.596	21.767	711.013	732.780	1.149
12	Oct. 18	0.397	4.94	1.023	0.627	0.093	1.557	12.2	10.6	1.557	21.237	711.013	732.250	1.149
13	Oct. 18	0.339	3.85	1.023	0.594	0.093	1.524	12.8	11.1	1.524	20.789	711.013	731.802	1.149
14	Oct. 18	0.293	3.05	1.023	0.572	0.094	1.501	14.4	12.2	1.501	20.478	711.013	731.491	1.149

small velocities would have been far different, if this curve had been determined from actual measurement of the losses.

Summary of Results.—The additional points, representing the relation of m to D, have been included in the logarithmic plotting on Plate LIX, and the letter D has been attached to all the diameter points to distinguish them from the velocity observations. The location of the result on the 48¾-in. pipe and that calculated for the 2 840.5 ft. of 31-in. pipe both fall very near the author's average line. Using these results in connection with those presented by Mr. Moritz, to the exclusion of those of the 4-, 12-, and 22-in. pipe, for reasons which he has given, a line was obtained the equation for which did not differ materially from the expression, $m = 8.6\ D^{-1.263}$. This appears to be almost conclusive evidence that the author's expression for m will apply closely to the majority of the wood pipe lines, as constructed and operated on the Sunnyside Project.

The writer thinks, however, that, in preparing a tentative formula for general use, all complete data, which can be accepted as criteria for the loss of head in wood pipe, should be recognized in arriving at a conclusion. Utilizing the results of Noble on the 44- and 54-in. pipes, of Marx, Wing, and Hoskins on the 72-in. pipe, and of Adams on the 14-in. pipe, and excluding the author's result for the 22-in. pipe and those derived by him for the single observations on the 18-, 24-, and 30-in. pipes, the writer has calculated by the method of least squares (equal weight being attached to all results), an expression to represent

ON 31-INCH CONTINUOUS WOOD STAVE PIPE.

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MERCURY GAUGES NOS. 2 AND 3.
Mercury Gauge No. 2 was used in Observations Nos 2 to 8, inclusive.
Mercury Gauge No. 3 was used in Observations Nos. 1, and 9 to 14, inclusive.

U.	L.	Mercury.	T° cent.	t° cent.	Mercury reduc.	Water equivalent.	Elev. of "L" zero.	Elev. of water column.	Length.	Loss of Head. Total loss.	Loss of Head. Per 1 000 ft.	Area.	Velocity.	n.
(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)	(24)	(25)	(26)	(27)	(28)	(29)	(30)
0.583	0.092	1.611	27.2	22.2	1.609	21.942	710.911	732.853	7 354	11.921	1.621	5.241	4.068	0.01136
0 239	0.064	7.374	24.2	21.6	7.369	100.435	637.182	737.267	4 513.5	7.439	1.648	5.241	4.028	0.01154
0 596	0.070	7.037	21.7	13.3	7.027	95.497	637.282	732.779	4 513.5	8.842	1.959	5.241	3.881	0.01283
0 678	0.072	7.117	13.6	12.8	7.117	96.721	637.282	734.003	4 513 5	12.008	2.661	5.241	4.589	0.01289
0.550	0.072	6.989	11.1	11.1	6.990	94.996	637.282	732.278	4 513.5	6.278	1.390	5.241	3.150	0.01322
0 581	0.072	7.020	14.0	11.7	7.018	95.376	637.282	732.658	4 513.5	6.082	1.347	5.241	3.125	0.01313
0 555	0.073	6.993	14.5	12 2	6.991	95.011	637.282	732.293	4 513.5	4.342	0.962	5.241	2.587	0.01332
0 550	0.074	6.987	14.2	12.8	6.986	94.944	637.282	732.226	4 513.5	3.418	0.757	5.241	2.211	0.01371
0 482	0.096	1.535	19.2	12.8	1.533	20.914	710.672	731.586	7 354	2.930	0.398	5.241	1.698	0.01292
0.469	0.097	1.521	17.2	12.8	1.520	20.739	710.672	731.411	7 354	1.962	0.267	5.241	1.311	0.01345
0.462	0.097	1.514	12.9	12.8	1.514	20.657	710.672	731.329	7 354	1.451	0.197	5.241	1.126	0.01329
0.451	0.095	1.505	9.7	10.6	1.505	20.533	710.672	731.205	7 354	1.045	0.142	5.241	0.943	0.01381
0.444	0.095	1.498	11.7	11.1	1.498	20.438	710.672	731.110	7 354	0.692	0.094	5.241	0.735	0.01339
0.438	0.095	1.492	13.9	12.2	1.492	20.356	710.672	731.028	7 354	0.463	0.063	5.241	0.582	0.01290

this relation of m to D. The result is the solid line of negative slope shown on Plate LIX, the equation of which is $m = 8.3\ D^{-1.20}$.

The arithmetical mean of the corresponding exponents of velocity is 1.78, so that the general expression becomes

$$H = \frac{8.3\ V^{1.78}}{D^{1.20}}.$$

The writer does not wish to be understood as advocating a rigid application of this equation in general practice for determining friction losses in wood pipe. It is derived from a limited number of experiments, and its use under conditions different from those met in those experiments might produce very discordant results. For this reason it seems advisable to apply limiting values to the constants in the above expression.

Referring again to the general plotting on Plate LIX, it will be seen that lines drawn through the 8- and 44-in. pipe results, parallel to the one already drawn, will represent fairly well the limits of the value of m. The equations of these lines are $m = 6.34\ D^{-1.20}$ and $m = 11.84\ D^{-1.20}$.

The variation in the exponent, n, will be considered as from 1.70 to 1.94, and the general formula becomes

$$H = \frac{6.34 \text{ to } 11.84}{D^{1.20}}\ V^{1.70 \text{ to } 1.94},$$

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H = the loss of head, in feet per 1 000 ft.,
V = the velocity, in feet per second,
and D = the diameter of the pipe, in inches.

This range of values is large, but the writer thinks it unsafe to narrow the analysis any further, owing chiefly to the wide variation in the results on the large sizes.

The only rational method, which suggests itself, of applying the above expression for purposes of design, is to make a study of the diagram and exclude from the analysis those results which were obtained under conditions radically different from the case in hand. It is hoped that more experiments will be added to this list, in order that the limits in the equation may be reduced to such values as will render the formula of more practical use to the engineer.

Mr. Darrach. CHARLES G. DARRACH, M. AM. SOC. C. E. (by letter).—This paper is an aid in the solution of the difficult problem of standardizing reliable formulas for determining the flow of water in conduits. The author's formula is unique in its simplicity and remarkable in its range of application. That all experimental observations do not adhere closely to an empirical formula is not unusual, but if such a formula is derived from many observations, idiosyncrasies and abnormal conditions are readily noted, and, by comparison with the standard, the cause and nature of the divergence may be indicated and the remedy applied.

The fact that the coefficient of roughness, n, varies inversely with both the velocity and the diameter or area of the pipe has been recognized for many years, but the writer is not aware that this element is embodied in any other formula. Similar conditions of wetted perimeter naturally produce greater retardation of flow in small than in large pipe, and it seems to be a well-established fact that the greater the velocity the less the roughness of the skin surface retards the flow.

Aside from the value of the experiments and formula in the determination of friction in wood pipes, they enable one to make a comparison with pipes of other material.

If velocities in new cast-iron pipe, computed from other formulas in use, are compared with those computed from the author's formula for wood pipe, abnormal results become apparent. Darcy's formula gives a greater velocity for a 4-in. cast-iron pipe with a slope of 0.01 than the author's wood pipe with the same slope. Kutter's formula (n = 0.012, slope = 0.0001) gives as high a velocity for a 60-in. iron pipe as for a 60-in. wood pipe by the author's formula.

Compared with the author's results for wood pipe, Kutter's results (n = 0.012, slope = 0.01) vary from 70% for 4-in. pipe to 80% for 16-in. pipe, which percentage continues to 60 in. diameter. The writer

has modified the author's $K = 8.6$ for wood pipe to $K = 13$ for clean cast-iron pipe, which decreases the velocity to 80% for all slopes from 0.01 to 0.0001. The details are shown in Table 15. Mr. Darrach.

TABLE 15.—Comparison of Velocities for Different Diameters, Calculated from Various Formulas.

Diameter, in inches.	Wood Pipe, Slope = 0.01.	Clean Cast-Iron Pipe, Slope = 0.01.					
	Moritz.	Moritz.		Kutter.		Darcy.	
	$K = 8.6$.	$K = 13.0$.	Percentage.	$n = 0.012$.	Percentage.		Percentage.
4	2.87	2.30	80	2.00	70	3.00	104
6	3.81	3.06	80	2.77	72.7	3.80	100
8	4.66	3.72	80	3.51	75.3	4.40	96.6
10	5.45	4.36	80	4.20	77 +	5.00	91.2
12	6.19	4.925	80	4.80	77.4	5.45	88
16	7.57	6.02	80	5.88	78	6.40	85
20	8.85	7.04	80	7.00	80	7.18	81 +
24	10.00	7.99	80	8.06	80.6	7.88	78.8
36	13.35	10.65	80	10.74	80.4	9.71	73.8
48	16.35	13.00	80	13.20	80.7 +	11.26	68
60	19.10	15.20	80	15.20	80	12.28	64.3 +
		Slope = 0.0001.					
6	0.296	0.234 +	80	0.226	77		
8	0.361	0.286 +	80	0.284	79		
10	0.422	0.335 +	80	0.350	83		
12	0.479	0.381 +	80	0.407 +	85		
16	0.587	0.466	80	0.520	88.6		
20	0.686	0.545	80	0.619	90.0		
24	0.779	0.620	80	0.707	91.2		
36	1.035	0.823	80	0.970	93.7		
48	1.260	1.006	80	1.200	95.2		
60	1.480	1.180	80	1.520	102.77		

The author's formula seems to be adaptable to the use of a coefficient, variable only with the degree of roughness of the wetted perimeter, making $K = 8$ for the smoothest pipes, and multiplying the results by a percentage for rougher pipe.

The carrying capacity of cast-iron pipes rapidly decreases with age and consequent incrustation, while, on the other hand, wood pipes, while maintaining a high carrying capacity, are subject to decay, and have a comparatively short life. Considering this high carrying capacity, there should be better inspection, and greater care in their manufacture, so as to prolong their period of usefulness.

Unless the wood is kept saturated with water, early decay may be expected, and the iron wrapping or bands should be of sufficient gauge to withstand a certain amount of rust. To be successful, the wood used should be of uniform and permeable texture, similar to well-

Mr. Darrach. seasoned, clear, white pine, cedar, or redwood, with the grain normal to the circumference. With this kind of timber, the staves can be thicker and the banding irons heavier and spaced farther apart.

The use of fir or other resinous woods should be avoided, except for temporary work or where cost makes the more porous woods inexpedient; and if resinous wood is used, there should be no knots and the staves should be sawed with straight grain normal to the circumference, and not parallel with it, as in general practice. The fat layers prevent saturation, and the pipes rot from the outside.

In the writer's experience, fir pipes laid in a sandy loam showed signs of serious decay in a short life of 5 years, and in one instance the metal banding was cut through by sand impinged upon it by a pin-hole leak. In fact, fir pipes laid under paved streets are looked upon with much disfavor.

Many years ago (about 1846) cement-lined water pipes were laid in Wilkes-Barre, Pa., and served that city for about 20 years. In 1887 the writer laid cement pipes (made by the American Pipe Manufacturing Company) in the village of Ridley Park, Pa., and has been informed by John W. Ledoux, M. Am. Soc. C. E., that in 1910 (23 years later) they were found to be in perfect condition. These pipes have a shell of thin wrought iron encased in a very fat cement mortar of natural or Rosendale cement; the surfaces are very smooth. In the writer's opinion, similar pipe have all the durability of cast iron, and maintain the high carrying capacity of wood stave pipe.

It is unfortunate that all the experimental data on the subject, obtained during the past 40 years, although apparently "replete with inconsistencies," are not co-ordinated, for, as Mr. Hering justly says: "The cause of all the trouble probably does not lie in the facts themselves, but in their interpretation"; and it is to be hoped that in this discussion all available information on the subject will be presented and formulated so that it may be referred to a special committee which would eliminate idiosyncrasies and interpret the facts in a series of standards, as the experiments of the author seem to indicate can be made.

Mr. Moritz. E. A. Moritz, Assoc. M. Am. Soc. C. E. (by letter).—Mr. Enger calls attention to a possible error of levels on the 22-in. pipe. His analysis purporting to show that there was a constant error of 0.14 ft. in the observed losses is not at all conclusive; and it is a significant fact that the solid black points in his diagram, which represent the two highest and the two lowest losses as determined by the writer, lie almost exactly on a straight line and this line represents the locus of all the points more closely than Mr. Enger's line represents the locus of all the open circles. Any conclusion based on his analysis, therefore, is not justified, and the fact that four of the points could be

swung into a straight line by the addition of a constant quantity is probably a coincidence only. Mr. Moritz.

The presence of air in the manometer tubes was watched very carefully throughout all the experiments. The writer recalls that in this particular case there was some trouble with air at the upper gauge, and no readings were taken without first turning on and releasing the pressure several times and making certain that the water ran up to practically the same point on the gauge each time. This can be considered as good evidence that no air column existed. As a further precaution against air bubbles, a small leak was maintained from the top of the pipe, several feet above the manometer, to release air bubbles. The probability of the presence of an air column of constant length, 0.14 ft., is exceedingly remote. The accumulation of air at summits, if there are any, seems to be a plausible explanation for some of the vagaries exhibited in the observations on this pipe, although the writer is not prepared to subscribe to this opinion.

The observations presented by Mr. Campbell are interesting, but they would be infinitely more valuable if more had been made and if more details were given. The observed discharges are somewhat larger than would be indicated by the writer's tests. The discharges, calculated from the formula presented by the writer, for the two pipes mentioned by Mr. Campbell, are 5.83 and 5.58 cu. ft. per sec. for the 10 and 16-in. pipes, respectively, as compared with 5.88 cu. ft. per sec., as measured, giving differences of 0.8 and 5.1%, respectively. Taken at its face value, this is a remarkable agreement. The small difference may be due to greater smoothness of the white pine pipe, but the data are not sufficient to establish this or any other facts in regard to his observations, and an intelligent comparison with others is impossible.

Some two years ago, a leading technical journal published an article by a contributor in which it was stated that tests on a wood stave pipe, 10 ft. in diameter, gave a value of 0.011 for Kutter's n. This statement interested the writer very much, and he made an attempt to obtain the details, some of which he succeeded in getting after some difficulty. Among other peculiar things, he discovered that the specific gravity of commercial mercury was taken as about 12.5 (the exact figure is not recalled), and there was no indication that this had been determined by a laboratory test. Inquiries were made as to why this figure was used, but no reply was received. Unsupported statements like the above are a menace to the advancement of hydraulic knowledge, and have not the slightest elements of value. It would be well if our engineering and other technical journals would scrutinize matter of this kind more fully, and refuse to publish it unless fully supported by scientific analysis or experimental data.

Mr. Moritz. As one editor of an engineering journal has truly remarked, "We allow too much that is false and misleading and incomplete to be perpetrated in cold type."

Although the paper is intended primarily as a contribution to hydraulic literature, and not to advocate a new formula, Mr. Hering's discussion calls for some remarks on the subject of the safest and most useful type of formula for general use. A single gauging—possibly affected by some unknown condition which would be brought to light in a series of observations—is of very little value for predicting the performance of a similar structure. This opinion is supported by a multitude of published data. It is much better, therefore, that the available data, for which full details are known, be considered in their entirety in arriving at a decision, rather than that a single observation or set of observations be used. Furthermore, it is best if these results are interpreted by those who are competent by virtue of their training, ability, and experience to do so. Mr. Hering's table of "experience data" is undoubtedly a very valuable and safe tool for the experienced hydraulician, but it is dangerous in the hands of an amateur. Mr. Hering himself voices this opinion when he says, "The cause of all the trouble probably does not lie in the facts themselves, but in their interpretation." Viewed in this light, the exponential type of formula with fixed constants is much better than Kutter's, and is a safer tool in the hands of the inexperienced. The writer agrees that the criticism that the Kutter formula is too complicated is not a serious one, although there is no doubt that the exponential type is simpler, and is better adapted to graphical use. The writer has the highest respect for the Kutter formula and the able engineers who deduced it, but he believes the day of its application to the flow of water in pipes is fast coming to an end.

Mr. Darrach is evidently not familiar with Western fir, else he would be more charitable toward its use in wood stave pipe. In the Northwestern States, Douglas fir is used very extensively. The writer knows of one pipe of this material which has been in the ground for about 15 years, and there are undoubtedly many others which have been in use as long or longer. He has not examined into the condition of this pipe, but there are no apparent leaks, and it continues to carry its full supply of water. During this period, also, it has been used only 7 months of each year, so that the conditions have not been conducive to a long life. The 14-in. pipe of these experiments has been in the ground about 5 years, and has also been alternately wet and dry. Several auger holes bored through the top near its upper end showed the staves to be in perfect condition. Occasionally, hemlock staves will get through the mill, and these rot out very rapidly; however, if Douglas fir is carefully selected and inspected, it makes an excellent and durable material for wood pipe.

Mr. Moritz.

The writer wishes to thank Mr. Moore for presenting the additional data on 48¾ and 31-in. pipes. These observations were not completed in time to be included in the original paper, and were presented by Mr. Moore, at the writer's request, with an independent discussion.

The results on the 48¾-in. pipe check the author's formula almost exactly, but the observations are too few to be convincing. The following circumstance in connection with this pipe should be mentioned: About a month after the observations were made, two leaks were discovered in the portion of the pipe under the river. The first evidence of these leaks was a drop in the gauge reading at the outlet, and on examination, two miniature "geysers" were found spouting up from the river bottom on the line of the pipe. Detailed examination showed that the silt-charged water, forced through a longitudinal joint between staves, had cut in two several of the §-in. round bands. There is not much doubt, therefore, that these leaks, although much smaller, were present at the time the observations were made. Two facts indicate that the leaks must have been very small at that time: one is that the operation records show no difference in the quantity entering and the quantity discharged by the pipe, and the other is the lack of evidence, during the observations, of any leak at the river, for it was very shallow and clear, and a leak of any considerable magnitude would have produced ripples and a turbid condition of the water. The experimenters passed over the line of the pipe in a boat repeatedly during the tests, and would have noticed such a condition. Whatever effect there was, due to these leaks, extended over considerably less than half the length of the pipe, as both leaks were in the half toward the lower manometer. The writer would estimate the total leakage as not more than 1 cu. ft. per sec., affecting one-half the length of pipe experimented on.

As there was no flow of water into the gauge after the full pressure had been reached, Mr. Moore's second explanation of the apparent discrepancy in Observation No. 2 cannot be true. If there was any pressure at all—which there apparently was—it must have been the full pressure. In amplification of Mr. Moore's explanation of the diameter measurements of the 31-in. pipe, it should be stated that the amount of swell of the staves from moisture was determined by soaking several samples in water for two weeks and comparing the thickness before and after this process. The thickness of the staves used in the pipe was very uniform.

The high losses indicated by the observations on this pipe are very surprising. The pipe was built of the best of materials, by an experienced pipe-builder, under the writer's direction, and he can vouch for the excellence of the construction. The slope of the velocity curve would not seem to indicate an abnormally rough interior. Several times, during construction and afterward, boys were discovered

Mr. Moritz. throwing stones into the pipe, and an accumulation of these might have a material effect. An examination of the interior of this pipe throughout its length would not be difficult, and it is hoped to have this accomplished soon, as it will undoubtedly throw some light on the subject.

Mr. Moore's analysis of all available data is not sufficiently comprehensive to warrant the formulation of an equation purporting to represent such data, but his efforts are properly directed and should be continued.

Attempts to place limiting values on the exponent of V, and on the factor, m, are only misleading, and the resulting expressions have neither practical nor academic value. If these were true limits, in the accepted sense of that term, they would be of some use, but, from the nature of their selection, they cannot be. The problem of selecting the proper form of expression for a particular case resolves itself into selecting one value for m and one exponent for V. The ability of the busy practicing engineer to do this intelligently has already been remarked upon. This is by no means a slur on the busy engineer's intelligence.

Mr. Moore says:

"The only rational method, which suggests itself, of applying the above expression for purposes of design, is to make a study of the diagram and exclude from the analysis those results which were obtained under conditions radically different from the case in hand."

Nothing could be more irrational than this, and as for the results in the diagram being "radically different" from any "case in hand," no such case could occur in wood pipe design, and any difference that might occur would be difficult to predict. Mr. Moore's hope that the limits in his equation will be reduced by the addition of other experiments will never be realized, as is evident from the manner in which these limits were selected.

Mr. Williams' objection to the units in the formula is not a serious one, as the transformation to any other units is accomplished by a simple and rapid calculation. From the nature of his request for additional discharge data, it is assumed that he refers to the 55¾-in. pipe. Reference to Fig. 3 and Plate LVII will throw much light on this subject. The side contractions of the weir are about 1 ft., but the concrete wing-walls of the intake structure make an angle of 120° with the weir, and this may have some effect on the direction of the water filaments. The cross-section, Fig. 3, indicates a bottom contraction of about 2.5 ft., but this was reduced to about 1 ft. by the silting up of the approach channel. No cross-sections of the approach channel were taken, but these can be approximated from the information presented, remembering that the concrete wing-walls of the intake structure make an angle of 30° with the center line of the approach

channel. The hook-gauge was attached to the left wing-wall (looking up stream), about 6 ft. up stream from the weir, and was in a stilling box, Fig. 22. The exact length of weir was 18 ft., as noted in Table 2, and the ends were perpendicular to the crest. A wye-level was used in transferring elevations from the weir crest to the hook-gauge. The hook was made especially heavy so that this could be done. The crest Mr. Moritz.

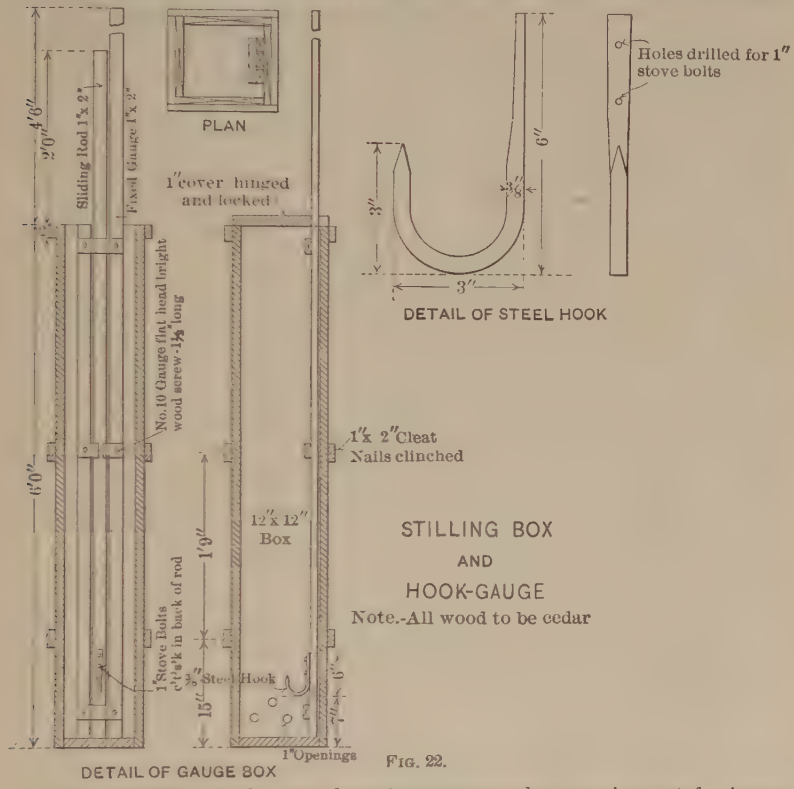

FIG. 22.

was not perfectly level, and elevations were taken on it at 1-ft. intervals, as follows:

0	3.287	10	3.282
1	3.282	11	3.275
2	3.280	12	3.276
3	3.282	13	3.270
4	3.280	14	3.268
5	3.276	15	3.268
6	3.276	16	3.270
7	3.276	17	3.276
8	3.283	18	3.288
9	3.282	Average	3.278

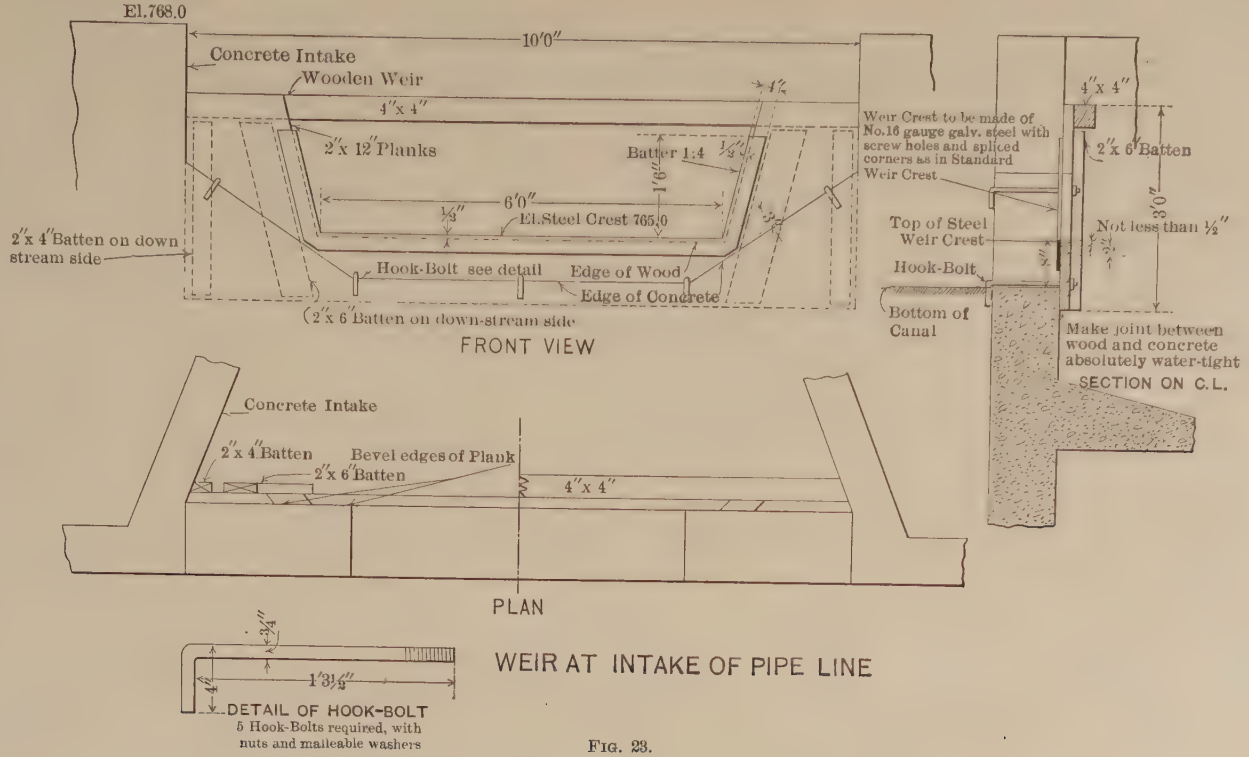

FIG. 23.

Mr. Moritz.

INTAKE TO 31-INCH PIPE.

Mr. Moritz.

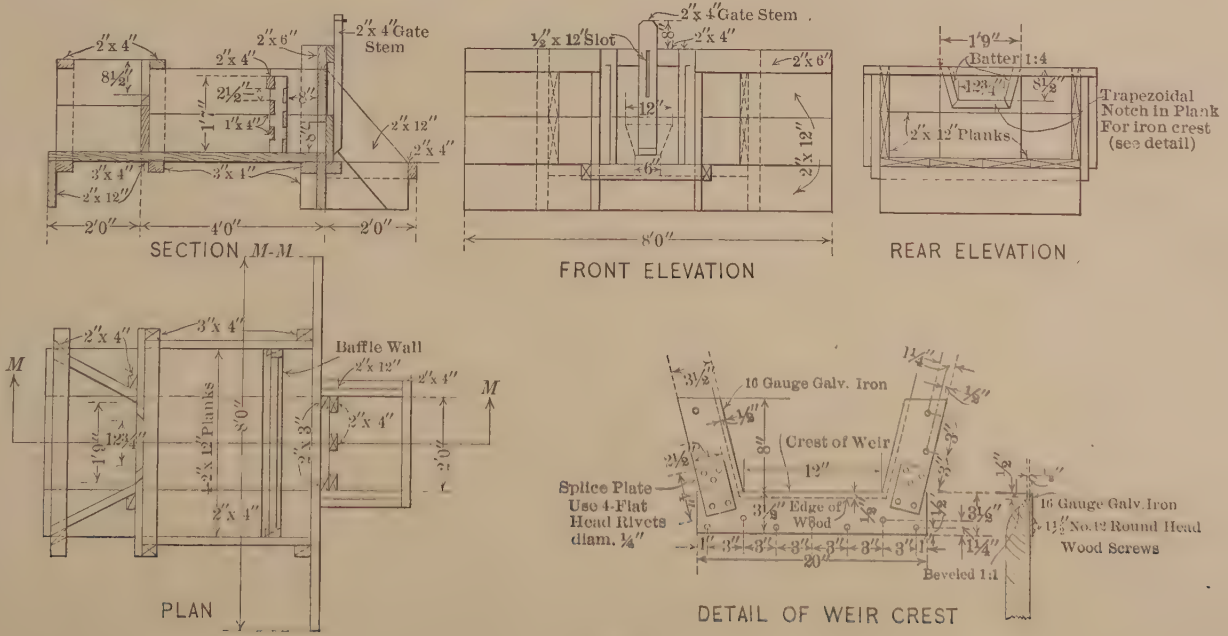

FIG. 24.

Mr. Moritz. The elevation of zero of the hook-gauge was 3.299, requiring a correction of — 0.021 (3.278 — 3.299) to be made to all observed gauge readings.

Supplementing further the discharge data already presented, the following is added: Mr. Moore has described the manner of measuring discharge quantities for the 31-in. pipe. The intake to this pipe is shown on Plate LX, the photograph having been taken soon after the construction was completed and before the weir had been placed. Fig. 23 shows the details of the weir. The hook-gauge was about 6 ft. up stream from the weir, and elevations were transferred with a wye-level. The weir was level, exactly 6 ft. long, and the sides had a true 1 to 4 batter.

The weirs for all the other pipes were either in pools or in wooden boxes. In all cases the velocity of approach was reduced to a negligible quantity by baffle-boards. Fig. 24 shows the details of the standard measuring box with a 1-ft. weir, as used on the Sunnyside Project. The dimensions of the boxes for larger weirs are correspondingly larger. The hook-gauge was generally attached to the side-wall at or near the middle of the box, and elevations were transferred from weir to gauge by a wye-level and checked by the water level at zero quantity. The baffle-walls shown in Fig. 24 are very effective in quieting the water, but where these were not considered sufficient, additional baffles were used. Where the weirs were in pools, the gauge was usually placed from 4 to 6 ft. up stream from the plane of the weir.

For the benefit of those who may want to go fully into the details of these experiments at some future time, the writer wishes to repeat his statement made in the paper, that a full and complete record of all observations and measurements is on file in the office of the United States Reclamation Service at Sunnyside, Wash., and will be available for examination at any time.

OCTAVE CHANUTE.

MEMOIRS OF DECEASED MEMBERS.

OCTAVE CHANUTE, Past-President, Am. Soc. C. E.*

Died November 23d, 1910.

This memoir records the professional career of an Engineer closely identified for the last sixty years with the development of transportation on land and in the air.

Octave Chanute was born in Paris, France, on February 18th, 1832. His father was Professor of History in the Royal College of France, in Paris, until 1838, when he accepted the appointment of Vice-President of Jefferson College, in the State of Louisiana. He was a resident of Louisiana until 1844, when he removed to New York City and engaged in literary pursuits. His son, Octave, who was six years old when the family came to the United States, completed his education in New York, and became, to use his own expression, thoroughly Americanized.

In 1849, at the age of seventeen, Mr. Chanute started work on the Hudson River Railroad, beginning, as was usual at that day, at the very foot of the ladder. He introduced himself to the Resident Engineer at Sing-Sing and asked for employment. When told that there was no vacancy, he asked for permission to serve without pay as a volunteer Chainman. This was granted somewhat reluctantly, with the result that within two months the young man was put on the payroll at $1.12½ per day, and thought his fortune made; nor was he far wrong, for Mr. Chanute often said that this was the only position he had ever applied for, and that he had been continuously engaged since that time without having had to solicit employment.

He remained with the Hudson River Railroad for four years, until the completion of construction, when he became Division Engineer, at Albany, in charge of terminal facilities and of maintenance of way.

In 1853 immigrants were pouring into Illinois to buy Government lands at $1.25 per acre (which are now selling at more than $50 per acre) and railroad construction was proceeding rapidly. Mr. Chanute went West with the late H. A. Gardner, M. Am. Soc. C. E., his former Chief Engineer on the Hudson River Railroad, and was engaged on the surveys and construction of what is now a portion of the Chicago and Alton Railroad, between Joliet and Bloomington. Before this work was quite completed, he became Chief Engineer of the eastern portion of what is now the Toledo, Peoria and Western Railroad, and built that road from Peoria to the Indiana State Line, a distance of

* Memoir prepared by the following Committee: Onward Bates, Past-President, Am. Soc. C. E., Robert W. Hunt, Charles F. Loweth, and Charles L. Strobel, Members, Am. Soc. C. E.

112 miles. Upon its completion, in 1857, he remained in charge of maintenance of way until 1861, when, the road having gone into the hands of a Receiver, he accepted the position of Division Engineer of Maintenance of Way of the Pittsburgh, Fort Wayne and Chicago Railroad, between Chicago and Fort Wayne. This appointment was given him by his old friend and Chief, Mr. Gardner, who one year later recommended him for the position of Chief Engineer of the reconstruction and maintenance of the Western Division of the Ohio and Mississippi Railroad, from St. Louis to Vincennes. This he accepted, but six months later the road changed hands, by one of those sudden vicissitudes not uncommon in those days, and Mr. Chanute, having by that time attained a reputation for industry, efficiency, and fidelity, received simultaneously several offers of employment. He accepted that of the railroad which he had first served in the West, and, in 1863, became Chief Engineer of the Chicago and Alton Railroad. As such, he took charge of the reconstruction which, by that time, had been found necessary, for these early American railroads were cheaply built. His duties included the maintenance of way and the building of an extension of the line from Alton to St. Louis.

During this connection, which lasted until 1867, Mr. Chanute submitted a competitive design for the Union Stock Yards of Chicago. His design was selected in preference to a score of others, and he was made Chief Engineer of the Yards, supervising their construction, in addition to his railroad duties.

These various engagements brought him in contact with prominent railroad men, and he was next offered the design and construction of the pioneer bridge over the Missouri River at Kansas City. This offer was accepted, and he tendered his resignation as Chief Engineer of the Chicago and Alton Railroad. The Board of Directors, in accepting his resignation, passed a resolution of regret at the severance of his connection with that company. The construction of the Kansas City Bridge, across a stream so rapid, shifting, and ill-reputed as the Missouri River, involved what were, at that time, a number of novel engineering problems. It was completed successfully in July, 1869, and attracted general interest as the first bridge built over the Missouri River. An account of the construction of the Kansas City Bridge was written by Mr. Chanute and the late George S. Morison, Past-President, Am. Soc. C. E., his Principal Assistant Engineer, and published in 1870.

While engaged in completing the Kansas City Bridge, Mr. Chanute was placed in charge of the building of several railroad lines extending into Kansas, which were planned to secure a portion of the cattle trade coming overland from Texas: The Kansas City, Fort Scott and Memphis Railroad, from Kansas City to the line of the Indian Territory; a parallel line, now known as the Southern Kansas Division of

the Atchison, Topeka and Santa Fé Railroad; a connecting line between these two; and a line from Atchison northward, the whole comprising the construction of about 400 miles of railroad, and, incidental thereto, the design and construction of the Union Stock Yards at Kansas City. These various works were completed in 1871, and Mr. Chanute then became General Superintendent of the Southern Kansas Railroad.

In 1873, when the Erie Railroad was reorganized, Mr. Chanute was made Chief Engineer. It was then proposed to double-track this road, to standardize its 6-ft. gauge, to extend the line to New England and to Chicago, and generally to expend about $50 000 000 in improvements. This promised well for the Engineer, but the financial panic of 1873 upset the arrangements made in England for funds. Less than $5 000 000 was expended on the road, which served, however, to double-track the main line, change the gauge to standard, and to improve the gradients, so that, during the ten years which Mr. Chanute spent on the Erie, the average number of cars in freight trains was increased from 18 to 35. For a time he was in charge of the motive power of the railroad in order to readjust the distribution of locomotives.

Upon his return to New York, in 1873, he observed that "Rapid Transit" in that city had been under discussion for nearly twenty years without any definite results. The necessity for more rapid communication was evident; many projects for superseding horse-cars had been proposed, but none was recognized as solving the problem, and the public seemed to be completely at sea. After some unsatisfactory inquiries as to the reasons for this condition of affairs, Mr. Chanute concluded that the question could be best settled by investigation through a committee of the American Society of Civil Engineers. He was made Chairman of such a committee, consisting of Messrs. W. N. Forney, Ashbel Welch, Charles K. Graham, and Francis Collingwood, which undertook to collect all the data and facts. These were contained chiefly in pamphlets, aggregating about 4 000 pages octavo, and, after this literature had been digested, communications were invited by circular. Five public meetings were held, to give hearings to all who chose to appear, interviews were had with organized bodies and with citizens who were likely to possess information, and a canvass was made of property owners and tenants along the proposed routes.

The resulting report was made public in 1875. It recommended substantially the plan subsequently carried out, that is, the building of four lines of elevated railroads along the avenues, to be operated by steam locomotives, and stated that such lines would be profitable, which was not generally believed by moneyed men at that time. At first, the report was assailed vigorously by men interested in other

projects, but the public accepted it, and it was followed almost immediately by the requisite legislation and the building of the roads.

In this investigation, the laboring oars were wielded by Mr. Chanute and Mr. Forney, then Editor of the *Railroad Gazette,* and they wrote the reports and appendices. The former, indeed, had done all his work at night, in order not to interfere with his railroad duties. He found himself so prostrated upon its completion that he had to take a vacation, and he went to Europe for four months to recruit.

In 1880, Mr. Chanute was appointed Chairman of a committee of the American Society of Civil Engineers to report upon wood preservation. The investigation occupied five years, and resulted in the publication of a report of great value, which was the authority on the subject for many years.* In 1885, a number of American railroads called on the Chairman to build plants for wood preservation, and, after that, he became interested in wood preservation as a business.

In 1883, he resigned from the Erie Railroad and removed to Kansas City, opening an office as Consulting Engineer. In this capacity he had charge of the design and construction of the iron bridges of the Chicago, Burlington and Northern Railroad, between Chicago and St. Paul. Later, he performed similar work for the Atchison, Topeka and Santa Fé Railroad on the extension of its line from Kansas City to Chicago, involving, besides a number of minor streams, the bridging of the Missouri River at Sibley and of the Mississippi River at Fort Madison.

As long ago as 1874, Mr. Chanute had become interested in aviation, but it was not until 1889, when he moved to Chicago, and thenceforth made that city his home, that he could find time to devote himself seriously to the solution of this "Problem of the Ages." With characteristic industry and thoroughness he engaged in an extensive correspondence with men all over the world who were interested in this subject, and gathered and systematized all information of importance which he could find, investigating the records of experiments of the past two hundred or three hundred years. The fruit of these labors appeared in a series of articles entitled "Progress in Flying Machines," first published in the *American Engineer and Railroad Journal,* New York (October 1st, 1891, and following issues), and republished in book form in 1894. This publication was of the greatest importance to the advancement of the art, for not only were all experiments of any importance described in detail and the principles elucidated, but the author's opinion on the causes of failure and the probable direction in which improvement might be expected, were stated.

At about this time Mr. Otto Lilienthal had been making successful gliding experiments near Berlin, and these induced Mr. Chanute to build a Lilienthal glider and to attempt experiments with man-carry-

* *Transactions*, Am. Soc. C. E., Vol. XIV, p. 247.

ing models in continuation of his previous experiments with small models. The site chosen was Dune Park, near the present Town of Gary, on the sand dunes of Indiana. His purpose was mainly to attain equilibrium in the air, and he hoped to accomplish this by adjustments which should be largely automatic. Unlike the unfortunate Lilienthal, who met his death in August, 1896, while making one of his gliding experiments, Mr. Chanute's many experiments—upwards of 200 flights—were free from any misadventure to life or limb. The Lilienthal glider was an unwieldy monoplane, requiring great skill in management. It was soon abandoned and a multiplane glider substituted, and this, in turn, was replaced by the much simpler and more efficient biplane, the prototype of the present Wright aeroplane. These experiments were described by Mr. Chanute in a paper entitled "Gliding Experiments," read October 20th, 1897, before the Western Society of Engineers, and published in its *Journal*.*

All these experiments and investigations were made in a genuinely scientific spirit and at his own expense, free to all who were interested, and conducted without any thought of pecuniary or other benefit to himself. He had at an earlier time described his efforts with characteristic modesty as:

"Giving much of his leisure to the investigation of the chances of success in the possible solution of the problem of Aerial Navigation, not with the expectancy of solving it himself, for he held this would be the work of many men by a gradual process of evolution, but with the hope of advancing the question a little, and making the process easier for those who came after him, by eliminating some of the causes of past failures and laying down the principles which will have to be observed."

On October 20th, 1909, just twelve years after the reading of his first paper on aviation before the Western Society of Engineers, he read his last paper, entitled "Recent Progress in Aviation," in which he described the bewildering record of successful flights which had been achieved, giving a complete chronology of aviation from December 17th, 1903, when the Wrights made the first successful man-flight in history, to October, 1909.

After Mr. Chanute's death, the Aero Club of Washington said of him:

"Lilienthal, Chanute, Langley, and Maxim are the four names that will ever be inseparably linked with the early stages of flying-machine development, the stages that preceded the successful invention of the first man-carrying machine by the Wright brothers. These four men elevated an inquiry, which for years had been classed with such absurdities as the finding of perpetual motion and the squaring of the circle, to the dignity of a legitimate engineering pursuit."

* Vol. II, p. 593.

Mr. Wilbur Wright in "Aeronautics" paid him the following tribute:

"If he had not lived, the entire history of progress in flying would have been other than it has been, for he encouraged not only the Wright brothers to persevere in their experiments, but it was due to his missionary trip to France in 1903 that the Voisins, Bleriot, Farman, De Lagrange, and Archdeacon were led to undertake a revival of aviation studies in that country, after the failure of the efforts of Ader and the French government in 1897 had left everyone in idle despair. Although his experiments in automatic stability did not yield results which the world has yet been able to utilize, his labors had vast influence in bringing about the era of human flight. His 'double-deck' modification of the old Wenham and Stringfellow machines will influence flying-machine design as long as flying machines are made. His writings were so lucid as to provide an intelligent understanding of the nature of the problems of flight to a vast number of persons who would probably never have given the matter study otherwise, and not only by published articles, but by personal correspondence and visitation, he inspired and encouraged to the limits of his ability all who were devoted to the work. His private correspondence with experimenters in all parts of the world was of great volume. No one was too humble to receive a share of his time. In patience and goodness of heart he has rarely been surpassed. Few men were more universally respected and loved."

Mr. Chanute was elected an active member of the Western Society of Engineers on July 12th, 1869; was President in 1901, and was elected an Honorary Member on January 5th, 1909. Some years ago he presented to that Society a fund of $1 000, the interest of which was to provide bronze medals to be awarded annually for the best papers on Civil, Mechanical, and Electrical Engineering subjects. It is noteworthy that the committee to award prizes for papers read to the Society, in 1909, reported, after Mr. Chanute's death, that his paper of October 20th, 1909, on "Recent Progress in Aviation" was the first in merit of all the papers submitted during that year, and the Society presented the medal to his family.

He was elected an Honorary Member of the British Institution of Civil Engineers on May 21st, 1895.

He was an Honorary Member of the Canadian Society of Civil Engineers, a Corresponding Member of the French Society of Civil Engineers, and also of the Chilean Society of Engineers.

He was a Member of the American Railway Engineering Association, the American Institute of Mining Engineers, the Society for the Promotion of Engineering Education, and the American Association for the Advancement of Science.

He was a Fellow of the American Aeronautical Society, an Honorary Member of the Aero Club of America, and was President of the Illinois Aero Club. He was also honored by foreign aero clubs.

The Aeronautical Society of Great Britain awarded him a gold medal in recognition of his distinguished services in promoting the art of aviation. He held the degree of Doctor of Engineering from the University of Illinois.

Mr. Chanute was Chairman of the Executive Committee of Engineering Societies which had charge of the International Engineering Congress at the World's Columbian Exhibition in Chicago, in 1893. He was a frequent contributor to the publications of scientific societies and to the various technical journals. A list of the various contributions to engineering literature, prepared by him, begins with his paper on "Pneumatic Bridge Foundation," published in the *Journal* of the Franklin Institute in 1868, and ends with his paper on "The Present Status of Aerial Navigation." This last paper was published in *Science* on December 29th, 1910. This list contains 60 titles, the two principal ones being his books on "The Kansas City Bridge" and "Progress in Flying Machines," which have been referred to.

In his relations with his fellow men, it is sufficient to record that he was unselfish, just, and kind. The present generation needs no eulogy of him, and, for posterity, it may be simply stated that he possessed the qualities of mind and heart which endeared him to his friends, caused those with whom he came in contact to respect and admire him, and which, ripened and chastened by the hard school of experience, produced in him one of our foremost Engineers, whose busy life was rich in works and achievement, and kindly and generous acts, commanding our admiration and prompting our love.

While traveling with his daughters in Europe in the summer of 1910 Mr. Chanute was taken ill with pneumonia. For a time his illness was of so serious a nature that a fatal termination was expected, but he recovered sufficiently to return to America, and, after a lingering illness, died at his home in Chicago, November 23d, 1910. His remains were interred at his old home in Peoria, Ill.

Mr. Chanute was married, in 1857, to Miss Anne Riddell James, of Peoria, Ill., who died in 1902. He is survived by a son and three daughters.

Mr. Chanute was elected a Member of the American Society of Civil Engineers on February 19th, 1868, and a Fellow on July 11th, 1872; he served as a Director for four years (1873-74-75 and 1885), and as Vice-President for two years (1880-81). He was President in 1891, and for the five succeeding years was *ex-officio* a member of the Board of Direction.

WILLIAM METCALF, Past-President, Am. Soc. C. E.*

DIED DECEMBER 5TH, 1909.

William Metcalf was born at Pittsburg, Pa., on September 3d, 1838. His father, Orlando Metcalf, was an eminent lawyer of his time in Allegheny County, and was in partnership with Mr. Loomis under the firm name of Loomis and Metcalf.

In the fall of 1854 Mr. Metcalf entered Rensselaer Polytechnic Institute, and was graduated with the Class of 1858. After his return from college he was engaged as Superintendent in the Fort Pitt Foundry, of which his uncle, Charles Knap, was the principal owner. In addition to constructing what was known in those days as heavy machinery, the Fort Pitt Foundry was engaged, before and during the Civil War, in casting guns, mortars, and shells for the United States Government, the largest cast-iron guns ever made in the world being made there during that time, notably two 20-in. guns weighing 80 tons each.

In 1869 Mr. Metcalf bought an interest in the old firm of Miller, Barr, and Parkin, which became Miller, Metcalf, and Parkin, the partnership continuing until 1889, at which time the firm was incorporated as the Crescent Steel Company. The business of this company was that of making high-grade crucible steel, for tools and many special uses where quality counted for much. Late in 1895 he sold out his interest in this company, and, until 1897, was not engaged in active business.

In 1897, he organized the Braeburn Steel Company, and began the construction of the plant at Braeburn, which was put in operation during the summer of 1898. The business of this company is that of making high-grade crucible carbon steel and also open-hearth steel. Mr. Metcalf was President of this company and active in its affairs to the time of his death.

During his career as a steel maker he made a very careful study of the metallurgy of iron and steel, and had a thorough knowledge of the methods of manufacture, uses and treatment of steel. His book, entitled "Steel, A Manual for Steel Users," is regarded as a standard work on that subject.

Mr. Metcalf's peculiar knowledge of steel and its properties, and of the processes of manufacture, made him a valuable witness in many important cases in which patents were being litigated, and he was frequently consulted as an expert in that line.

In addition to his business career, he was a member of the Board of Arbitration which appraised the property of the Monongahela Navi-

* Memoir prepared by the following Committee: Thomas H. Johnson, George Westinghouse, Jesse W. Walker, Emil Swensson, and Thomas Rodd, Members, Am. Soc. C. E.

Yours sincerely
Wm Metcalf

gation Company and awarded to that company more than $3 000 000. This award was accepted by both the company and the Government as a fair appraisal, and was paid by the United States for the property franchise of the company.

Mr. Metcalf was one of the founders of the Homeopathic Hospital, of Pittsburg, and was its President until his death. He was a life-long member of St. Peter's Protestant Episcopal Church, and was its Senior Warden for about thirty-five years.

He was prominently identified with the organization of the Engineers' Society of Western Pennsylvania, was its first President, and, on January 20th, 1880, read the first paper presented to that Society, choosing for his subject "Why Does Steel Harden?"

Mr. Metcalf was elected a Member of the American Society of Civil Engineers on July 2d, 1873. He served as a Director during 1883-1884, and as President in 1893. In the latter year, by virtue of his office as President of the Society, he presided at the meetings of the Engineering Congress held at Chicago during the Columbian Exposition.

He was also a member of the following Societies: American Institute of Mining Engineers, American Society of Mechanical Engineers, American Iron and Steel Association, Institution of Civil Engineers of Great Britain, and the Engineers' Society of Western Pennsylvania.

In his death the Society has lost a valued member, who cherished a deep interest in its welfare, and whose eminent abilities gave force and vigor to those of its discussions in which he participated. He was an earnest and true friend, wise in counsel, charitable in judgment, kindly, with sympathy for all who appealed to him, and universally beloved by all whose privilege it was to know him intimately.

JOHN BOND ATKINSON, M. Am. Soc. C. E.*

DIED SEPTEMBER 21ST, 1911.

John Bond Atkinson was born on November 11th, 1840, on his father's farm near Mullica Hill, N. J. He was the eldest son of George T. and Elizabeth B. Atkinson, who were leading members of the Society of Friends.

In 1863 he was graduated from the Polytechnic College of Pennsylvania as a Bachelor of Civil Engineering. From 1863 until April, 1866, he was engaged with the Pennsylvania Railroad Company as Assistant Engineer of Construction and Maintenance of Way on the Philadelphia and Erie Railroad. His next work was on the west end of the Hoosac Tunnel, as Engineer and Superintendent for B. N. Farren, F. Am. Soc. C. E., the contractor. At this time he built 430 ft. of arching and masonry through the "soft rock," which work was undertaken and finished after the failure of the Commonwealth management to make progress. From April, 1869, until April, 1871, he was Engineer and builder of Turner's Falls Branch of the Vermont and Massachusetts Railroad. He also aided in the construction of Atlantic Avenue, Boston.

In April, 1871, Mr. Atkinson went to Kentucky and became Engineer and Manager for the St. Bernard Coal Company, later named the St. Bernard Mining Company. He held various positions in this company, and finally became President in 1894. For many years he was regarded as one of the best-informed coal men in America, and a mining engineer and executive officer of unsurpassed ability.

He took a leading part in exploiting Kentucky's mineral, forestry, and agricultural products and resources at the World's Columbian, the Atlanta, the Louisiana Purchase, and the Jamestown Expositions. He was Financial Director and President of the Kentucky Jamestown Exposition Commission.

In the first conference of Governors, in the White House, at Washington, in May, 1908, Mr. Atkinson took part, as one of the advisors of the Governor of Kentucky, the Hon. Augustus E. Willson, and presented one of the most important papers, "Forestry, as Related to Mining Interests." In Governor Willson's speech before that conference he paid Mr. Atkinson a high and deserved tribute, which appeared as follows in the official proceedings:

"The oratorical product of Kentucky ended some years ago, and we are now studying strictly utilitarian phases of life. We have in Kentucky many important coal companies. One of those has been managed in a manner that I think in its business management and in its own foresight presents a lesson of usefulness to every State. It

* Memoir prepared by the Secretary from papers on file at the House of the Society.

was mentioned here that in the production of coal it had cost about seven lives for every million tons. This company, in ten years, has produced 1,100,000 tons, with only one life lost. (Applause.) This company bought great areas of land. A State Commissioner of a great State mentioned that they had planted a million young trees. This one company has alone planted on its land one million walnut trees and about a quarter of a million locusts, catalpa, and poplar or tulip trees, in the past few years. (Applause.) This information is from a paper, which I shall submit, by Mr. John B. Atkinson, one of my associates and advisors here, and a very useful one, who perhaps gave me the only good thought that Kentucky could bring here."

Mr. Atkinson was an authority on practical forestry, as he had been actively engaged in this work, in Hopkins County, Ky., for twenty years, in which time he had planted millions of young trees, many of them having been grown from seed.

Mr. Atkinson's first wife was Sarah Ann Black, of Mt. Holly, N. J., to whom he was married on May 23d, 1867. She died in November, 1900. They had two children, John, who died at Swarthmore College in 1883, while nearing the completion of his education, and Mary Newbold, who became the wife of Paul M. Moore and died in 1907, leaving two sons. On September 16th, 1903, Mr. Atkinson married again, his second wife being Anna Newbold Black. She and their little daughter, Nan, four years of age, survive him. Two brothers also survive, George C. Atkinson, of Earlington, Ky., who has been associated with him in business for forty years, and is now Vice-President of the St. Bernard Mining Company, and Warren Atkinson, a farmer, of Mullica Hill, N. J.

Mr. Atkinson was an indefatigable worker. He possessed an excellent physique, strong and commanding will power, quick and sure judgment, and great nervous energy. His interests were so broad and so diverse that the twenty-four hours of the day were all too short in which to compress the things he did for the business interests he represented, for his many employes, for his friends, and for humanity. He was always a strong supporter of schools and worthy educational movements. He influenced his company to give substantial help to the schools of Earlington, two modern, common graded and high school buildings having been built and equipped entirely at its expense. The company has also contributed annually to their support. He also aided many other educational and moral undertakings. One of his good works is represented by a school that bears his name, Atkinson College, at Madisonville, for the practical and industrial education of negro youth, which school he has largely supported. It was due to his influence that prohibition was established in Earlington in 1882, and his fighting determination has made it a fixed policy to this day.

Mr. Atkinson's health began to fail early in 1910. The disease

from which he suffered, which rarely occurs, is known to medical science as lymphatic leukæmia, and has never been cured. He died on September 21st, 1911, at his country home, near Wrightstown, N. J., where he had spent the summers for several years. His body is buried in West Laurel Hill Cemetery, in Philadelphia.

Mr. Atkinson was elected a Member of the American Society of Civil Engineers on September 5th, 1877. He was also a member of the Engineers' Club of New York, of the Engineering Association of the South, of the American Association for the Advancement of Science, and of the Pendennis Club, of Louisville.

GEORGE JOSEPH BELL, M. Am. Soc. C. E.*

DIED OCTOBER 6TH, 1911.

George Joseph Bell was born at Mossband Farm, at the head of the Solway Firth, in the Parish of Kirkandrews-on-Esk, Cumberland, England, on August 9th, 1835. He was educated at Croft House, Brampton. After leaving school he worked on large surveys for the Firth Commissioners, for commutation purposes, and on private estate surveys, with his uncle, the late Robert Bell, of The Nook, Irthington, Carlisle, England, who was a friend of George Stephenson and a well-known Civil Engineer and Surveyor.

After his uncle's death in 1856, Mr. Bell carried on the business until June, 1867, when he was appointed District County Surveyor of the Brampton Highway Board, his ancestors having lived and owned estates in the Brampton District for more than 600 years. While in this position he built many miles of new roads and stone bridges.

In 1871, Mr. Bell was elected District Surveyor of the Highway Board for the Isle of Axholme, in the Lindsey Division of Lincolnshire. He held this office until January, 1879, when he was appointed, from more than 200 applicants, County Surveyor and, later, County Surveyor and Bridge Master, for Cumberland, England, which position he held until his sudden death on October 6th, 1911—a period of 33 years. While in these positions Mr. Bell built, under the direction of the Highways and Bridges Committee, many miles of new roads through difficult country, laying out a new road over Sty Head Pass. He also designed and constructed a number of bridges.

Mr. Bell was an authority on oblique or skew arches for bridges, and published a work entitled "Segmental and Elliptical Oblique or Skew Arches," which has since become a standard on the subject. He constructed several bridges in Cumberland on this principle.

For some time prior to his death, Mr. Bell had been in failing health, but was able to attend to his duties, and it was a great shock when he was suddenly taken ill and died almost immediately while sitting in his chair in his office. He is survived by one son and two daughters.

Mr. Bell was a Member of the Institution of Civil Engineers, the County Surveyors' Society, of which he was a Past-President, the Municipal and County Engineers' Association, and the Sanitary Institute; he was also a Freemason.

Mr. Bell was elected an Associate Member of the American Society of Civil Engineers on November 2d, 1898, and a Member on October 1st, 1902.

* Memoir prepared by the Secretary from information furnished by Percy Wilson, Esq., and from papers on file at the House of the Society.

ful engineers, but was of such a retiring disposition that he never attained the high position to which his ability and experience entitled him. He was highly esteemed by his employees and by all who knew him. His death is a loss, not only to the Profession in Canada, but to his many friends.

Mr. Bell was a Member of the Institution of Civil Engineers, of Great Britain, and was elected a Member of the American Society of Civil Engineers on June 4th, 1884.

CHARLES MORTON EMMONS, M. Am. Soc. C. E.*

DIED SEPTEMBER 14TH, 1911.

Charles Morton Emmons was born in Hartland, Hartford Co., Conn., on January 11th, 1866. With his mother, he moved to Huron, Wayne Co., N. Y., where, later, he lived with Mr. Joseph Chapin and his wife.

At the age of eighteen Mr. Emmons successfully passed the examination for a free scholarship at Cornell University, and was graduated from the Engineering Department in the Class of 1888. Immediately after his graduation, he entered the employ of the Penn Bridge Company, at Beaver Falls, Pa., as Draftsman. He remained with this Company until 1892 when he was appointed Inspector with the United States Engineer Corps, at Buffalo, N. Y., and was employed in supervising the Government work in Buffalo Harbor. In 1897 he was transferred to the U. S. Engineer's Office at Erie, Pa., and, as Junior Engineer, was employed in connection with the Government work in Erie Harbor until 1901. In 1901 Mr. Emmons returned to the Penn Bridge Company as Estimator and Designer, and, later, he was given entire charge of this Department.

On his return to Beaver Falls he took an active interest in the Y. M. C. A., and was President of the Board of Directors for a number of years. He was also active in the Presbyterian Church of Beaver Falls, having been a Trustee for many years; at the time of his death he was an Elder, and also Superintendent of the Sunday-school.

Mr. Emmons' death, which occurred on September 14th, 1911, at the Beaver Valley General Hospital, was caused by an internal cancerous growth. He is survived by his widow, Anna K., and an adopted son, Charles Curtis Emmons. The funeral services were held at the First Presbyterian Church, at Beaver Falls, on September 16th, and the interment was made in Forest Lawn Cemetery, at Buffalo, on September 17th, 1911.

Mr. Emmons was elected a Member of the American Society of Civil Engineers, on April 1st, 1903.

* Memoir prepared by C. H. Vaughan, M. Am. Soc. C. E.

WILLIAM BION EWING, M. Am. Soc. C. E.*

DIED APRIL 8TH, 1911.

William Bion Ewing was born at Williamsburgh, Pa., on June 21st, 1858. Just prior to the Civil War, his father, William Ewing, with his family, moved to Huntington, Ind., where he was in business as a merchant for more than 40 years. His mother, Anna Ruehl, was a daughter of John Ruehl, of Williamsburgh, Pa., an engineer on the construction of the Brooklyn Bridge under the late John Roebling, M. Am. Soc. C. E. William Bion Ewing was taken to see this bridge when he was quite young, and at once declared his determination to become a Civil Engineer.

During his boyhood he kept this idea before him, and whatever money he could make was saved to help him attain his ambition. Money was not too plentiful in those days, but he persevered year after year, and at last was able to enter Cornell University, from which he was graduated, in 1883, with the degree of Civil Engineer. He worked during his vacations on the Toledo, Delphos, and Burlington Railroad, and practically paid for his own education.

After his graduation Mr. Ewing was employed by the Department of Public Works of New York City on the hydrographic survey of Croton Lake and the location of the Aqueduct.

In 1886 he went to Chicago as Assistant Engineer for the Chicago and Northwestern Railway Company, and, for two years, was engaged in locating new lines in Michigan and Minnesota. During this time he also did some bridge work in connection with his surveys.

In 1888 he was employed by the Union Stock Yards Company, Chicago, Ill., and had charge of the construction of various warehouses, viaducts, buildings, pavements, and railroad yards.

In 1889 Mr. Ewing again became Assistant Engineer for the Chicago and Northwestern Railway Company, locating and building tracks and yards in and around Chicago.

From 1890 to 1891 he designed and constructed new water-works at Cazenovia, N. Y., and Litchfield, Conn., the work at Cazenovia including two dams for storage reservoirs. In 1891, he designed the foundation and superintended the erection of two 90-ft. girders, and designed and constructed a filtration well for an additional water supply at Huntington, Ind.

In 1892, he became Chief Engineer of the Chicago Heights Land Association and, until 1895, was employed in designing and laying out the Village of Chicago Heights and in building railroad yards, putting up factories, putting in macadam pavements, and inaugurat-

*Memoir prepared by George H. Bremner, M. Am. Soc. C. E.

ing a system of water-works at that place. He also designed and superintended the construction of many of its important industrial plants.

From 1895 until his death Mr. Ewing was engaged in general consulting engineering practice in Chicago and its vicinity. He was City Engineer for a number of cities and villages near Chicago, among them being: La Grange, Dalton, Burnham, Duquoin, and Riverdale. During this time he gained many friends and established a wide reputation in the practice of civil, sanitary, and municipal engineering. He designed the water-works systems for Cazenovia, N. Y., Duquoin, Chicago Heights, La Grange Park, Melrose Park, and Summit, Ill. He also designed sewerage systems for La Grange, La Grange Park, Chicago Heights, Wheaton, Geneseo, Dixon, Barrington, Palatine, Arlington Heights, Dalton, Riverdale, Burnham, West Hammond, and Lyons, Ill., and Greensburg, Ind. At the time of his death he was building a large outlet sewer for La Grange, Ill. In addition to these important works, Mr. Ewing had charge of the installation of underground pneumatic tubes for the *Chicago American* and the *Chicago Tribune,* in Chicago.

In 1900 he assisted Desmond FitzGerald, Past-President, Am. Soc. C. E., in his investigation and report on the Chicago Drainage Canal, and measured and made soundings over the entire length of the Chicago River and the Drainage Canal from Lake Street, Chicago, to the dam at Lockport, Ill. During the same year, he also made measurements of all the railroads in the State of Michigan. In 1898 and 1902 he was engaged on preliminary surveys for the Monon Railroad.

On April 5th, 1893, Mr. Ewing was married to Mary E. Crandal, daughter of Edward M. Crandal, of Stevensville, Pa., who, with one daughter, survives him.

His death occurred from heart failure, while on a train on his way to inspect a sewage disposal works which he was constructing at La Grange. This was the first plant of the sprinkling filter type in Illinois.

Mr. Ewing possessed a genial, kindly nature, readily making acquaintances, and these his ability and integrity soon transformed into friends. Wherever he went he brought sunshine and good nature. He lived an upright, honorable, kindly life; his monument is in his work, but his memory will long survive in the hearts of his friends.

He was elected a member of the Western Society of Civil Engineers in 1891. He was also a member of the La Grange Suburban Club, La Grange Lodge No. 777, A. F. and A. M., and the Palisades Park Association, of Covert, Mich.

Mr. Ewing was elected a Junior of the American Society of Civil Engineers on April 3d, 1889, an Associate Member on February 3d, 1892, and a Member on November 2d, 1898.

HORACE JOSEPH HOWE, M. Am. Soc. C. E.*

DIED JANUARY 21ST, 1911.

Horace Joseph Howe was born at Lynn, Mass., on January 2d, 1860. He was graduated from the Medford High School in 1875, after which he entered the Massachusetts Institute of Technology, graduating in 1879, with the degree of S. B. Immediately after his graduation he entered the service of the New York, Lake Erie, and Western Railroad, serving as Rodman, Inspector, and Draftsman. He remained with this company until July, 1881, when he went West and secured employment in the Construction Department of the Northern Pacific Railroad on the design of wooden bridges, shops, stations, etc., in Minnesota. In 1882, he was appointed Bridge Engineer on the Missoula Division, with headquarters at Missoula, Mont.

In July, 1885, when this work was completed, Mr. Howe came East and, until April, 1886, was employed as Inspector of Bridge Materials on the St. Louis and San Francisco Railway, under the late James Dun, M. Am. Soc. C. E., Chief Engineer. He was also engaged on bridge work under the late George S. Morison, Past-President, Am. Soc. C. E.

In April, 1886, Mr. Howe re-entered the service of the New York, Lake Erie, and Western Railroad, and was employed, until April, 1894, in the Maintenance of Way Department, mostly on the Delaware and Susquehanna Divisions, and also as Roadmaster on about 200 miles of the New York, Pennsylvania, and Ohio Railroad, a leased line.

In September, 1894, he was appointed an Assistant Engineer with the Boston Transit Commission, on preliminary surveys, testing, estimating, designing, etc. He remained with the Commission until 1897, when the work on the Subway was completed. He was then appointed Assistant Engineer on construction work at Forest Hills, for the New York, New Haven, and Hartford Railroad. He was also engaged for a time as Assistant Engineer on the New York Central and Hudson River Railroad.

In 1900 Mr. Howe became an Assistant Engineer for the Rapid Transit Railroad Commission, and, later, for the Public Service Commission, of New York City. At the time of his death, he was Assistant Division Engineer, of the Fourth Division, First District, and his work in connection with the construction of the New York Subway, namely, the Manhattan Valley Viaduct, the West Side Viaduct and Extension, the Van Cortlandt Park Extension, and in the moving of the old and replacing of the new Harlem Ship Canal Bridges, on which he was Resident Engineer, showed his ability in his profession. He

* Memoir prepared by Theodore Belzner, Assoc. Am. Soc. C. E.

presented a paper on the latter subject entitled "Notes on the Replacing of the Superstructure of the Harlem Ship Canal Bridge,"* before the Society on January 5th, 1910. Another valuable paper, entitled "Some Instances of Piles and Pile-Driving, New and Old,"† by Mr. Howe, was presented before the Boston Society of Civil Engineers on March 30th, 1898. He was also an occasional contributor to the professional discussions of this Society, as well as to those of other engineering societies.

Mr. Howe was a gentleman of kindly disposition and remarkably even temperament. He was loyal to his friends and kind and considerate to his subordinates. His ability as an engineer and his value as a citizen, were recognized by all his associates.

He was a Member of the Boston Society of Civil Engineers, the Municipal Engineers of the City of New York, the National Civil Service Reform Association, and the Technology Club of New York. He was actively interested in church work, and was Treasurer of the Unitarian Club of Yonkers. He was also a member of the Yonkers Choral Society, having served as an officer for many years.

In 1899, Mr. Howe was married to Miss Stella S. Weston, of Medford, Mass., who, with three sons and a daughter, survives him.

Mr. Howe was elected a Junior of the American Society of Civil Engineers on May 2d, 1888, and a Member on March 2d, 1898.

* *Transactions*, Am. Soc. C. E., Vol. LXVII, p. 1.

† *Journal*, Assoc. of Eng. Societies, Vol. XX, p. 257.

WILLIAM JACKSON, M. Am. Soc. C. E.*

DIED JUNE 30TH, 1910.

William Jackson, the son of Samuel and Mary Wright (Field) Jackson, was born in Brighton, Mass., on March 13th, 1848. He was educated at the public schools of Brighton and the Massachusetts Institute of Technology, where he took the full course with the Class of '68, but, on May 4th, 1868, having obtained an appointment on the staff of the City Engineer of Boston, Mass., he left the Institute without receiving his degree. At that time the Institute had not conferred any technology degrees, and it is possible that their value was not appreciated by the youth of twenty to whom an engineering position, with assured remuneration, may have seemed to be more attractive. Later in life, however, it was one of Mr. Jackson's regrets that he had not remained at college and taken his degree.

His first work was on the construction of the Chestnut Hill Reservoir of the Boston Water-Works. In 1870 he became Engineer for the Town of Brighton, and also engaged in private practice until Brighton was annexed to Boston in 1873, when he again entered the office of the Boston City Engineer. During the next three years he was engaged on various work, including surveys for the introduction of water into Brighton and West Roxbury. From 1876 to 1885 he was Assistant Engineer on the Boston Main Drainage Works, a very important and difficult engineering undertaking.

In April, 1885, on the death of the City Engineer, the late Henry M. Wightman, M. Am. Soc. C. E., Mr. Jackson was appointed City Engineer, which position he occupied continuously until his death. In addition to his civic duties he was engaged in other important engineering works at various times. From 1887 to 1891 he was Chief Engineer for the Harvard Bridge Commissioners; from 1896 to 1900, Chief Engineer of Charlestown Bridge; and from 1898 until his death Chief Engineer for the Cambridge Bridge Commission. In the fall of 1898, in company with Edmund M. Wheelwright, Consulting Architect to the Cambridge Bridge Commission, he visited Europe in order to make a study of notable bridges preparatory to designing a monumental structure for Cambridge Bridge.

In 1891 and 1892 he was a member of the Boston Rapid Transit Commission; and a member of the Boston Statistics Commission from 1898 until his death. From 1902 to 1904 he was a member of the special commission on the abolition of grade crossings in Attleboro, Mass., and at his death had been for three years a member of commissions on the abolition of grade crossings in Foxboro, Westwood,

* Memoir prepared by the Secretary from papers on file at the Society House, and from a memoir prepared for the *Technology Review* by Frederic H. Fay, M. Am. Soc. C. E.

Canton, Sharon, and Mansfield, Mass. He was Consulting Engineer to the Cambridge Water Board on the construction of the Hobbs Brook Conduit, in 1904; to the Shore Road Commission, in Brooklyn, N. Y., in 1896 and 1897; and to the Massachusetts Harbor and Land Commission on the Commonwealth Dock, South Boston, in 1899. He was also a member of the Approving Board appointed by the Legislature in 1907 to pass upon plans for the development and extension of the drainage systems of Boston. On several occasions, when a vacancy occurred at the head of another city department, Mr. Jackson was appointed to fill the position until a permanent head could be chosen.

Mr. Jackson was a man of high ideals and notable ability. He was modest and unassuming, eminently fair in his dealings with others, and faithful to every duty entrusted to him. To those who were in close association with him, he was a true friend and a lovable man. He won the regard and hearty support of his subordinates, and inspired confidence and respect in all who knew him. The following, from an editorial in the *Boston Herald,* is representative of the tribute paid to him by the press:

"The record of a life spent in the service of the public is in itself the eulogy of City Engineer William Jackson. From boyhood to death he was a public servant, filling one post of duty after another, meeting every responsibility, great or small, preferring public service to the greater emoluments which he might have earned in private enterprise. He was a patriot, even though he shouldered a tripod instead of a musket."

On April 27th, 1886, Mr. Jackson was married to Miss Mary Stuart MacCorry, of Boston. Mrs. Jackson died on March 27th, 1905. One son, William Stuart Jackson, survives.

At the time of his death Mr. Jackson was a member of the following organizations: Union, Art, and Technology Clubs, of Boston; Boston City Club, Point Shirley Club, Boston Driving Club, Strollers' Club of New York, Allston Golf Club, Commonwealth Riding Club, the Masonic Fraternity, Boston Chamber of Commerce, Technology Alumni Association, Society of Arts, American Association for the Advancement of Science, National Geographical Society, Bibliophile Society, National Municipal League, American Civic Alliance, American Civic Association, New England Historical and Genealogical Society, Bostonian Society, and the Society of Colonial Wars. He was also a member of the Boston Society of Civil Engineers, and of the New England Water Works Association.

Mr. Jackson was elected a Member of the American Society of Civil Engineers on September 3d, 1884, and served as a Member of the Board of Direction from 1902 to 1904.

EDWARD MARION KENLY, M. Am. Soc. C. E.*

DIED JUNE 10TH, 1911.

Edward Marion Kenly was born on June 15th, 1867, at Ritchie Mines, W. Va., where his father, Major Kenly, was Superintendent and Chief Engineer, and, later, Chief Engineer, for many years, of the Baltimore City Water-Works. Mr. Kenly was educated at the Waverly High School, of Baltimore, and at Baltimore College.

He began his professional career in 1885 as Rodman for the Baltimore City Water-Works. In September, 1887, he was appointed Resident Engineer on the Lynchburg and Durham Railroad, which position he held until August, 1888, when he returned to the Baltimore City Water-Works as Assistant Engineer.

In March, 1889, Mr. Kenly went West to the State of Washington and engaged in private practice. He was also Townsite and Topographical Engineer for the Gray's Harbor Land Company.

In the spring of 1891, Mr. Kenly returned to Baltimore, where he was employed as Assistant Engineer, in the City Commissioner's office, in charge of the construction of the Read and Eden Streets sewers. In March, 1892, he was appointed Assistant Supervisor of the grading and masonry for a second track, on the Norfolk and Western Railroad. He remained in this position until June, 1893, when he engaged in private practice in the vicinity of Baltimore.

In the spring of 1894, Mr. Kenly again entered the employ of the Baltimore City Water-Works, as Instrumentman, which position he held until September of the same year, when he was appointed Engineer in charge of surveys and construction on the Baltimore, Middle River, and Sparrow's Point Electric Railroad.

In October, 1895, he was appointed Engineer in charge of improvements at Bay View Asylum, Baltimore, where he designed and built a garbage crematory, an ice manufacturing plant, and a sewerage system. From June, 1896, to February, 1898, Mr. Kenly was engaged in building water-works systems at Hanover, New Oxford, and Littlestown, Pa., and at Toneytown, Md.

In February, 1898, he went to Alaska and engaged in private practice, in the vicinity of Dawson and Forty-Mile, until November of the same year.

In August, 1899, Mr. Kenly became Resident Engineer on the Clearwater Short-Line Railway, in Idaho, and between that time and May, 1900, he also built a water-works system at Crisfield, Md.

In May, 1900, he went to West Virginia as Engineer and Superintendent of the mines controlled by William H. Gorman, of Balti-

* Memoir prepared by N. A. Gilman, Master Mechanic, Yakima Valley Transportation Company, North Yakima, Wash.

more, remaining in this position until June, 1905, when he returned to the West as Superintendent of the Montana Coal and Coke Company's mines, at Electric, Mont. He left this position in October, 1906, and went to Seattle, Wash., where between that time and the latter part of December of the same year, he was engaged in confidential work for the Harriman Lines in Western Washington.

In December, 1906, Mr. Kenly removed to North Yakima, Wash., and was employed as Chief Engineer of a proposed electric railroad which, later, he developed into the Yakima Valley Transportation Line. Here he remained as Chief Engineer and Superintendent until his death on June 10th, 1911. In swinging off a street car, Mr. Kenly ruptured the mesentery, and, although an operation was performed in an effort to save his life, it was of no avail. He is survived by his widow, his seventeen-year old daughter, and his one-year old son.

Mr. Kenly had the faculty of gaining the good will and confidence of all with whom he was associated, and was liked by every one. He was devoted to his family and his profession.

Mr. Kenly was elected a Member of the American Society of Civil Engineers on May 4th, 1904.

ALFRED COURTNEY LEWERENZ, M. Am. Soc. C. E.*

DIED MAY 27TH, 1911.

Alfred Courtney Lewerenz was born in Detroit, Mich., on September 24th, 1870. In his early youth he received instruction under Professor Hailleman, one of the earliest exponents in the United States of Froebel's ideas, and now a well-known author and educator. After some further preparatory work in the public schools of Detroit, Mr. Lewerenz entered the University of Michigan from which he was graduated with the degree of Bachelor of Arts, in 1891, and Bachelor of Science in Civil Engineering, in 1892, completing both courses in a total of four years, with an exceptionally brilliant record.

His first engineering experience was in the capacity of Draftsman with the Detroit Bridge Works, in 1892. Later in the same year he accepted a position as Chief Draftsman with the Chicago, West Michigan, and Detroit, and Lansing and Northern Railroads. In January, 1894, he became Draftsman and Assistant Civil Engineer with the Buffalo, Rochester, and Pittsburgh Railroad, being engaged on surveys, plans, and estimates for extension of line, improvement of grades and alignment, and plans for car shops, depots, and coal-handling apparatus.

In October, 1898, Mr. Lewerenz entered the Government service as Draftsman, under the Bureau of Yards and Docks, Navy Department, Washington, D. C., and was engaged on the designs of the new masonry dry docks at the Boston and the Mare Island Yards, and on various projects involving steel piers, wharves, and other water-front improvements. In March, 1899, he won a commission in the Corps of Civil Engineers of the Navy, with the rank of Lieutenant, Junior Grade, after passing, with a high mark, one of the most severe technical examinations then known.

Mr. Lewerenz's first duty after entering the Navy was as Assistant to the late Peter C. Asserson, M. Am. Soc. C. E., then Civil Engineer, U. S. N., at the New York Navy Yard. He was given entire charge of the construction of brick and steel storehouses, concrete quay walls, yard railways, and designs for new buildings. His tour of duty in the capacity of Assistant was unusually short, owing to the degree of ability he so early displayed and to the small number of officers then available. In June, 1899, he was ordered as Officer in Charge of the Naval Station, at New London, Conn. This detail was a very responsible one for a young officer, and there is no question but that he was selected for this position by M. T. Endicott, President, Am. Soc. C. E., at that time Chief of the Bureau of Yards and Docks,

* Memoir prepared by Leonard M. Cox, M. Am. Soc. C. E.

U. S. N., because of the ability he had shown during his service in Washington as a Civilian Assistant. While at New London, Mr. Lewerenz had entire charge of the construction of the coal storage and handling plant at that station, a very considerable amount of dredging work, and a new wharf project.

In February, 1903, he was ordered as Senior Civil Engineer in charge of all engineering work at the Navy Yard, Puget Sound, Wash., one of the most important of our larger naval stations. He was responsible for expenditures on public works amounting to about $500 000 a year, and for the design of many works of improvement, including a coaling plant, wharves, dredging, fire-proof shop buildings, storehouses, water-works, sewers, and electric equipment.

Mr. Lewerenz continued on duty at Puget Sound until November, 1907, when he was ordered to the Navy Yard, at Mare Island, Cal. Here he successfully completed the dry dock and was in charge of all civil engineering work during one of the busiest periods of the up-building of this yard. In July, 1910, he was sent to the Hawaiian Islands to take charge of the largest work of navy-yard construction under the Bureau, the building of the new naval base at Pearl Harbor. This work involved the layout and design of a complete naval station, including dredging to cost approximately $3 500 000, a dry dock costing more than $2 500 000, buildings and warehouses, streets, water-front improvements, water-works, sewers, and many other classes of construction. He was engaged on this duty and was carrying it on with his usual expeditious methods and excellent judgment at the time of his death.

Mr. Lewerenz was promoted to the rank of Lieutenant in April, 1906, and in October, 1909, attained the rank of Lieutenant Commander. In the course of his duty on the Hawaiian station, he contracted the malady which ultimately caused his death. For some months he made every effort to overcome the trouble, with the hope of continuing on duty, but his condition did not improve, and in the spring of 1911, he was ordered to the Mare Island Hospital, and there placed on the retired list. He died on May 27th, 1911, at the residence of relatives, in Detroit.

In the death of Mr. Lewerenz, the United States Navy, and especially the Corps of Civil Engineers, loses a most valuable officer. The splendid work that he accomplished at every station to which he was detailed, was acknowledged by all his brother officers, as well as by those members of the civil engineering profession at large who were familiar with it. His record was an enviable one, and gave promise of an exceptionally bright and useful future. He was a man of excellent judgment, sound professional knowledge, and great executive ability.

By nature, Mr. Lewerenz was gentle and lovable, gaining his triumphs in great part by the unconscious influence he exerted over all with whom he came in contact. He was highly esteemed by his superior officers, and popular with those under his direction. Among the officers of his own corps, his loss is mourned as that of a brother and friend. His unfailing cheery disposition, his unselfishness, his great strength of character, and his ready sympathy, as well as his fine and wholesome personality, are traits which will long be remembered.

Mr. Lewerenz was married on October 21st, 1906, to Miss Harriet Elizabeth Speir of Albany, N. Y., who, with their one son, Alfred C. Lewerenz, Jr., survives him. Their home life was delightful, and those who were fortunate enough to enjoy their hospitality were impressed by the love and Christian character that pervaded it.

Mr. Lewerenz was elected a Junior of the American Society of Civil Engineers on March 5th, 1895, an Associate Member on June 6th, 1900, and a Member on March 6th, 1906.

BENJAMIN FRANKLIN MACKALL, M. Am. Soc. C. E.*

DIED APRIL 5TH, 1911.

Benjamin Franklin Mackall was born at Baltimore, Md., on April 5th, 1857. He was the son of General William W. Mackall, a distinguished officer of the United States and Confederate Armies. His mother was Miss Aminta Sorrel, a daughter of Francis Sorrel, of Savannah, Ga.

Mr. Mackall was graduated second in the Class of 1877 from the Virginia Military Institute. From 1877 to 1879 he read law and was Principal of the Harrisonburg Academy, Rockingham County, Va.

He began his professional career in 1880 as Principal Assistant Engineer on the construction of the New Inlet Dam, of the Cape Fear River Improvement, in North Carolina, under the direction of Mr. Henry Baker and the late William P. Craighill, Past-President, Am. Soc. C. E.

From 1885 to 1888 Mr. Mackall was engaged on location and as Resident Engineer on the construction of the Philadelphia Branch of the Baltimore and Ohio Railroad under Major H. T. Douglass, then Chief Engineer; and from 1888 to 1890 on location and as Division Engineer in charge of construction and tunnel work on the Shawnee and Muskingum River Railroad.

From 1891 to 1892, he was employed as Engineer in charge of the location and plans for the National Highway from Washington, D. C., to Mount Vernon, under the direction of Peter C. Hains, M. Am. Soc. C. E.

From 1892 to 1896, Mr. Mackall was engaged in private practice in Washington, D. C., as a member of the firm of Mackall and Clark. During this time he located and built the Glen Echo Railroad, the Tenallytown and Bethesda Railroad, and did other suburban engineering work.

From 1896 to 1899, he was employed as Engineer on the Chesapeake and Ohio Canal, and from 1898 to 1900, as Division Engineer, he had charge of the location of several lines of railway for the Seaboard Air Line Railroad.

In 1900 Mr. Mackall was promoted to the position of Assistant Chief Engineer of the Seaboard Air Line Railroad, with headquarters at Portsmouth, Va., and was actively engaged in both location and construction work during the formative years of this railway system.

In 1905 he resigned his position with the Seaboard Air Line Railroad and engaged in private practice as Consulting Engineer, in

* Memoir prepared by W. W. Gwathmey, Esq.

Norfolk, Va. He died at his old home, Langley, Va., on April 5th, 1911.

Mr. Mackall was never married. He was a man of retiring disposition and modest manner. His ability and efficiency, his strict integrity, and his high standing among his associates as an engineer, commanded the respect of those who knew him, both in professional and private life.

Mr. Mackall was elected a Member of the American Society of Civil Engineers on March 6th, 1901.

CHARLES ARTHUR MATCHAM, M. Am. Soc. C. E.*

DIED SEPTEMBER 22D, 1911.

Charles Arthur Matcham, son of Charles and Elizabeth Matcham, was born at Torquay, Devonshire, England, on January 15th, 1862. He was educated in the schools of Hambledon and Brighton, England.

In 1875, Mr. Matcham went to London and, while employed during the day, attended the evening classes at an engineering school. At the public examinations he won first prize and honorable mention for mechanical drawing and designing at the Royal Academy of South Kensington, England.

From 1877 to 1880, he worked as a Mechanical Draftsman, in London. In 1880 he entered the employ of the International Bell Telephone Company, of New York, as Electrical Engineer. He remained in this position until 1881, during which time he designed and laid out construction work for telephone exchanges in Charleroi, Belgium, and in Riga, St. Petersburg, Warsaw, Moscow, and Odessa, Russia.

In the fall of 1881, Mr. Matcham came to the United States, and settled in Chicago, Ill., where he entered the employ of the Chicago Telephone Company, and was engaged in designing and building its exchanges. Later he removed to Memphis, Tenn., where he rebuilt the telephone exchange. In 1884, he came East to Allentown, Pa., and for several years managed the exchange there for the Pennsylvania Telephone Company.

In 1890 Mr. Matcham engaged in the business of cement manufacture, being associated with Mr. Thomas D. Whitaker in the Whitaker Cement Company, at Whitaker, N. J. He was General Superintendent of this plant until 1893, when it was sold to the Alpha Portland Cement Company, Mr. Matcham being retained as General Superintendent.

In 1897, with Colonel H. C. Trexler, he organized the Lehigh Portland Cement Company, of which he was Superintendent and General Manager. He built several mills for this company in Lehigh County, Pennsylvania, and contributed in no small degree to its prosperity.

In 1906 Mr. Matcham assisted in the organization of the Fuller Engineering Company, of Allentown, Pa., and became its President, a position which he retained until 1910. He was elected General Manager of the Allentown Portland Cement Company in 1908, and, later, Vice-President, retaining this office for two years during the building of the plant.

In 1910 failing health caused Mr. Matcham to retire from active business. He had taken a heavy cold, which settled on his lungs, and in

*Memoir prepared by H. F. Bascom, M. Am. Soc. C. E.

the hope of recovery he spent the spring and summer of 1911 at his summer home at Pocono Pines, in the mountains of Pennsylvania. His condition, however, became so serious that he was removed to his home in Allentown, Pa., where he died.

In 1893, when Mr. Matcham first became interested in the cement industry, the yearly output in the United States was less than 1 000 000 bbl.; at the time of his death this had increased to about 70 000 000 bbl.

The secret of Mr. Matcham's success was his executive ability, his judgment and control of men, and his faculty of getting from them the best of which they were capable. The most prominent of the younger cement engineers of the present time were his co-laborers, and the reduction of the problems of this important manufacture to an actual and exact practice, has resulted from his good judgment in the selection of his associates.

He also displayed great ability in solving the mechanical and chemical problems of cement manufacture, which involved not only the features of grinding and burning the material, but the equally vital ones of labor-saving and mechanical handling. The mills erected under his supervision were fully up to the most advanced practice at the time each was completed.

Some of the more prominent features of cement manufacture due to Mr. Matcham's inventive genius are: A cooler for clinker, a dryer for coal, a dryer for raw material, and a system of feeding pulverized fuel to the kilns, all of which have proved of great value and assistance to the manufacturer. The rotary dryer is largely used in the more important mills in the United States, in the dry process of cement manufacture, and has greatly increased the production. The coal dryer has become a standard method of drying coal previous to its pulverization for fuel use; and while the system of feeding pulverized fuel to cement kilns, devised by Mr. Matcham, was not generally successful, its use brought about a closer study of kiln conditions and may fairly be considered one of the important steps forward in the industry.

Mr. Matcham was married to Miss Margaret Ormrod, a daughter of Mr. and Mrs. George Ormrod, who, with three children, survives him. He was a Republican in politics, and a member of Grace Episcopal Church, of Allentown, Pa.

He was also a member of the American Institute of Mining Engineers, the American Society for Testing Materials, the National Geographical Society, the Manufacturers' Club of Philadelphia, the Livingston Club, of Allentown, and the Pomfret Club, of Easton, Pa.

Mr. Matcham was elected a Member of the American Society of Civil Engineers on June 5th, 1901.

ALBERT JOHN SCHERZER, M. Am. Soc. C. E.*

DIED MARCH 3D, 1908.

Albert John Scherzer was born at Annville, Pa., on June 5th, 1848. He was graduated as a Bachelor of Civil Engineering at the Polytechnic College of Philadelphia, in the class of 1866.

Mr. Scherzer commenced his professional career with the Boston, Hartford and Erie Railroad, serving as Rodman and Topographer during 1866 and 1867. In 1868 he was employed as Rodman and Section Engineer on the construction of the Dutchess and Columbia Railroad in New York State. In 1869 he was appointed Topographer and Senior Assistant on Surveys and Location of the Connecticut and Western Railroad, and from 1870 to 1872 was Assistant Engineer of Construction on the same road. During this time Mr. Scherzer served under the celebrated Engineers, the late O. W. Barnes, M. Am. Soc. C. E., of New York, and Mr. W. F. Shunk, of Pennsylvania. He remained a life-long friend of Mr. Shunk who dedicated his well-known field book, entitled, "The Field Engineer," "To His Old Comrade and Dear Friend in Token of Esteem for His Professional Attainments and Respect for his Manly Character."

At about this time Mr. Shunk was offered an important position in Colombia, South America, to take charge of railroad surveys. Being unable to accept the position, he recommended Mr. Scherzer who immediately started for South America to begin his Spanish-American life, and this extended over a period of 36 years.

Soon after his arrival in Colombia Mr. Scherzer was invited to go to Costa Rica, Central America, as Division Engineer of the Atlantic Division of the Costa Rica Railroad. In a short time he was made Chief Engineer and Superintendent of the Division and remained with the road until 1877. There are a number of engineers still living who shared in the hardships and privations encountered in the building of that difficult piece of railroad work.

In 1877 Mr. Scherzer went to Guatemala, Central America, as Chief Engineer and General Superintendent on Surveys, Construction and Operation, of the Guatemala Central Railroads, running from the port San José to Escuintla, thirty miles inland.

During the construction of this branch Mr. Scherzer kept parties in the field for the extension of the road to Guatemala City, the capital of the Republic, a distance of forty miles, which road was built on the lines located by his surveys.

Mr. Scherzer remained in Guatemala until 1882, when he became associated with the railways of Salvador, Central America, as Chief

* Memoir prepared by Franklin A. Snow, M. Am. Soc. C. E.

Engineer and Contractor. This line began at the Pacific Port of Acajutla and extended inland 12½ miles to Sonsonate. It was the beginning of steam railroads in the Republic of Salvador, and it was there that the writer first met Mr. Scherzer and remained with him for six years on the surveys and construction of the railroad to Sonsonate and its extension to Santa Ana, sixty-four miles inland.

Mr. Scherzer was a most careful and painstaking engineer, and his maps and notebooks were works of art. When the Intercontinental Railway Commission was making surveys for railroads to connect the United States with South America through Central America, all Mr. Scherzer's railroad plans for both Guatemala and Salvador were incorporated, and as separate maps they can be seen among the numerous sheets embodied in those extensive surveys.

Due credit was given him by the engineers in charge of the work, as shown in the following quotation:*

"At Santa Ana, Salvador, it was our good fortune to fall in with our countryman, A. J. Scherzer, Esq., whose reputation as a skillful and successful railroad engineer has extended all through Central America. He offered us freely and without reserve all the information at hand concerning the railroad work on which he was engaged, and later sent to Washington complete plans and profiles showing the *location* of the Santa Ana Railway as now built, enabling Mr. Anton Karl to prepare Map IX, which shows the line as now operated and its connection with our own surveys. I take this opportunity of extending to Mr. Scherzer my thanks for the information given and for hospitalities tendered myself and other members of our party during our stay in Santa Ana."

On page 125 of the Report, in reference to the Santa Ana Railway, it is stated:

"This was not an easy road to construct; on the contrary, careful study and skillful engineering were required to successfully overcome topographical difficulties; nor was it an easy matter to secure the necessary capital, the unsettled state of affairs in Central America making foreign investors reluctant to risk their money even in enterprises known, in themselves, to be meritorious and profitable.

"But Santa Ana was fortunate in possessing as a resident Mr. A. J. Scherzer, who by his energy, technical training, wide experience as a contractor, thorough acquaintance with the Central American people, and general knowledge of affairs, was able to fill the complex rôle of grantee, engineer, contractor, and financier, and by patient perseverance to push the work to a successful conclusion. * * * the Santa Ana Railway was thrown open to traffic July 27th, 1896, amid much rejoicing * * *."

Mr. Scherzer was General Manager of the Salvador Railways until May, 1906, when, owing to his ill health, he was compelled to resign.

* "Report of Surveys and Explorations Made by Corps No. 1, Intercontinental Railway Commission," Vol. 1, Pt. II, p. 7.

He remained in Salvador until 1907, during which time he surveyed and built the branch road running from Sitio del Niño to San Salvador, the capital of the Republic. He was also interested personally and financially in the building of the Sonsonate and Santa Ana Electric Light Systems and the Santa Ana Public Market.

He died at Santa Clara, Cal., on February 3d, 1908, and was buried in Cypress Lawn Cemetery, in the County of Santa Clara.

Mr. Scherzer was elected a Member of the American Society of Civil Engineers on January 6th, 1886.

JAMES SMITH, M. Am. Soc. C. E.*

DIED FEBRUARY 20TH, 1911.

James Smith was born at Row, Dumbartonshire, Scotland, on September 9th, 1872. From 1887 to 1891 he was an articled pupil and assistant to Daniel Macalister, M. Inst. C. E., at Greenock, Scotland, and was engaged principally on designs and estimates for dock and harbor works. In 1892 he entered the employ of Messrs. P. and W. MacLellan, Ltd., Engineers and Contractors, and remained with this firm until his death.

From January to August, 1892, Mr. Smith was Resident Engineer on the removal and reconstruction of Bilston Viaduct, on the North British Railway. The old viaduct was taken down and removed, new masonry piers, abutments, and approaches were built, and more than 700 tons of steelwork were erected, without interfering with through railway traffic. From August, 1892, until May, 1896, he was engaged in designing and estimating on general engineering work, and had practical experience in the steel car and bridge building workshops of the Clutha Works of Glasgow.

From May, 1896, to May, 1901, he was Resident Engineer at the Waverley Station, North British Railway, Edinburgh, supervising the contract work for Messrs. P. and W. MacLellan, Ltd., which included the removal of footbridges, approaches, and roofing of the station, and the reconstruction of the entire system, together with new foundations and a new system of drainage. The whole of this work, covering an area of about 12 acres, was done without interfering with existing railway or street traffic.

From May to October, 1901, Mr. Smith was engaged in designing and estimating on a number of engineering projects, among which were two swing bridges for Egypt, granaries and elevator towers for Buenos Aires, station extension for the Caledonian Railway at Glasgow, and widening at College, Glasgow, for the North British Railway. In October, 1901, he was sent to Mexico to report on the erection of a system of steel wharves for the Port of Coatzacoalcos (Puerto Mexico). In December of that year he returned to Glasgow, and was there engaged for the next six months in inspecting work for the Iquitos jetty on the River Amazon, and also on the preliminary work for the Coatzacoalcos wharves.

In December, 1901, Mr. Smith was married to Miss Anne Lawson Turnbull, eldest daughter of Mr. George Turnbull, Headmaster of the Bewick Schools, Howden, Newcastle-on-Tyne. There were no children.

*Memoir prepared by the Secretary from information furnished by Sidney Smith, Esq., and from papers on file at the House of the Society.

In July, 1902, he was appointed Manager and Chief Engineer on the construction of steel wharves and warehouses at Coatzacoalcos, and warehouses at Salina Cruz, for Messrs P. and W. MacLellan, Ltd., as contractors for Messrs. S. Pearson and Son, Ltd., in connection with the latter firm's agreement with the Mexican Government. These works comprise 4 630 ft. frontage of steel wharves and six double warehouses, each 416 by 106 ft., at Coatzacoalcos, and three double warehouses of the same size at Salina Cruz.

Mr. Smith remained in Mexico until late in 1908 and then returned to Scotland. In August, 1909, he was sent to India, by his firm, as Resident Engineer on the construction of the Manharpur Light Railway for the Bengal Iron and Steel Company. This railway from Manharpur to Notu, passes through thick jungle, in very heavy, hilly country, and had to be built during the dry season. While prosecuting this work, Mr. Smith was attacked repeatedly by jungle fever, and, though his firm advised him to take matters easier, he persisted in his endeavor to complete the road within the specified time. Finally, in December, 1910, his medical advisers ordered him home if he wished to save his life. He arrived in London on January 1st, 1911, though, during the voyage, both he and his wife were seriously ill. They went to St. Margaret's-on-Thames, but, by this time, malaria had weakened Mr. Smith's constitution so much that he contracted acute tuberculosis of the lungs, and died on February 20th. He was buried in Cathcart Cemetery, Glasgow, his funeral being attended by two of the Directors of Messrs. P. and W. MacLellan, Ltd., by which firm he was held in the highest esteem. Mr. William Turner MacLellan, Chairman of the Board, to whom he owed all his training as an engineer, was one of his stanchest friends. The Directors of the Bengal Iron and Steel Company sent a beautiful wreath bearing the inscription: "An able and devoted servant who placed duty before personal consideration." A more fitting epitaph could not have been written of him.

Mr. Smith was gentle, kind, and generous, but, nevertheless had an inflexible will, and there is no doubt that over-zeal in the interest of his employers hastened his death, which ended a career full of bright prospects.

An illuminated address, from the members of his staff in India, engrossed on silk and enclosed in a silver casket, was received on February 22d, just two days too late for him to see.

James Smith was elected an Associate Member of the American Society of Civil Engineers on November 1st, 1905, and a Member on March 31st, 1908.

WILLIAM DANA TAYLOR, M. Am. Soc. C. E.*

DIED AUGUST 26TH, 1911.

William Dana Taylor, the son of William Thomas and Mary (Hubbard) Taylor, was born in Montgomery, Ala., on January 22d, 1859. He was graduated from the Alabama Polytechnic Institute in 1881, with the degree of B. E., and in 1885 received the degree of C. E. He also did post-graduate work at Johns Hopkins, Cornell, and Chicago Universities.

During his professional life, extending over thirty years, he held many positions of great trust, and in none did he fail to give satisfaction.

From 1882 until 1886, Mr. Taylor was employed on surveys for the Mexican Central, in Mexico, and as a teacher in public schools in Alabama. In 1886, he was appointed Resident Engineer on the construction of a portion of the Birmingham Mineral Railroad, an important and extensive subsidiary of the Louisville and Nashville Railroad in the mineral district of Alabama. His ability soon caused his appointment as Chief Engineer of Construction of all the Louisville and Nashville lines then building in Alabama, which position he retained until the completion of the construction in 1891. He quickly acquired a reputation as an engineer who succeeded in getting work completed within the contract time.

From 1891 to 1898, the long period during which railway construction in the South was at a standstill, Mr. Taylor filled the Chair of Civil Engineering at the Louisiana State University. Resigning that position in 1898, he re-entered the more active engineering life as Chief Engineer of the St. Louis, Peoria, and Northern Railroad, which was planned as a new competitor for St. Louis and Chicago traffic. Shortly after the surveys for this line had been completed and the construction begun, its purchase by the Chicago and Alton Railroad Company caused its abandonment and the transfer of Mr. Taylor to the Engineering Department of the Alton, as Assistant Engineer. His most important work in this position was the reconstruction of the Missouri River Bridge, at Glasgow, Mo., and the surveys for track elevation in Chicago.

In 1901, Mr. Taylor resigned his position to become Professor of Railway Engineering at the University of Wisconsin, where he remained until February 1st, 1906, when he was appointed Chief Engineer of the Chicago and Alton Railroad. When the control of the Alton passed to the Toledo, St. Louis and Western Railroad, Mr. Taylor was the only general officer (with one exception) who was retained by the new management. Later in 1907, he became Chief Engineer of the Toledo, St. Louis and Western Railroad, and, in 1909,

* Memoir prepared by J. E. Willoughby, M. Am. Soc. C. E.

Chief Engineer of the allied lines, the Minneapolis and St. Louis and the Iowa Central. At the time of his death he was Chief Engineer of the Chicago and Alton and of the Toledo, St. Louis, and Western.

In addition to these positions, and sometimes in connection with them, Mr. Taylor held other important assignments. He was Captain of the Third United States Volunteer Engineer Corps during the Spanish-American War in 1898. In 1901, he made an appraisal of the physical properties of some of the Lake Superior ore-carrying roads for the Minnesota Railroad and Warehouse Commission; and, in 1902, obtaining leave of absence from the University of Wisconsin, he, as Chief Engineer, located the Knoxville, La Follette and Jellico Railroad (a subsidiary line of the Louisville and Nashville), from Jellico, Tenn., through eighty miles of difficult country, to Knoxville, Tenn. A paper entitled, "The Location of the Knoxville, La Follette, and Jellico Railroad, of the Louisville and Nashville System,"* containing a complete description of this work, was presented by Mr. Taylor before the Society on March 16th, 1904.

From 1903 to 1906, he acted as Expert Engineer for the State Board of Assessors of Wisconsin, and directed the re-valuation of the railways of that State—a notable achievement. He also made reconnaissances in the Continental Divide, was in demand as an expert witness in lawsuits involving the determination of engineering questions, and performed various other professional work.

Mr. Taylor was a frequent contributor to the engineering press, and to the publications of the various societies of which he was a member. His papers were exact and forcible, and gave evidence of vigorous thought. In the classroom, and in his more active life, his teaching was clear and was given with such kindness that he built up for himself, among the student body at his universities and among the young men in his employ, lasting friendships which added much to his pleasure and to the comfort and advancement of those who were associated with him. One of his former students writes:

"The grateful memory of his personality cheered and steadied me, and I have often said that W. D. Taylor was the most utterly unselfish man that I had ever met."

Imbued with the puritanism of the Presbyterian Church, of which he was a member, Mr. Taylor lived after its standards. In 1884, he was married to Miss Mary Ross, of Auburn, Ala. His second wife was Miss Annie L. MacIntyre, of Le Grand, Ala. Three children were born of each marriage.

Mr. Taylor was elected a Member of the American Society of Civil Engineers on March 1st, 1899. He was also a Member of the American Railway Engineering and Maintenance of Way Association, the Western Society of Engineers, the Society for the Promotion of Engineering Education, and of the Chicago Engineers' Club.

* *Transactions*, Am. Soc. C. E., Vol. LII, p 467.

CHARLES WIDNEY WOOD, M. Am. Soc. C. E.*

DIED APRIL 3D, 1911.

Charles Widney Wood was born in Pittsburg, Pa., on November 4th, 1863. He was the son of Captain B. D. Wood and Miriam A. (Widney) Wood. In 1866 his parents moved to New Orleans, La.

Mr. Wood was graduated from the Louisiana State University, after which he attended Rensselaer Polytechnic Institute at Troy, N. Y., from which he was graduated in 1884 with the degree of Civil Engineer.

From 1884 to 1888 Mr. Wood served in several capacities, including that of Assistant Engineer for the Birmingham, Ala., Bridge and Bolt Works, but his health failing he was forced to resign and go to Colorado, where he remained for about a year.

On his return to New Orleans, he entered the firm of B. D. Wood and Sons. About this time he also engaged in the wreckage and salvage business, to which he devoted most of his time and in which he became highly expert. Among his notable accomplishments in this line of work was the salving of the Schooner *J. Frank Seavy,* which had been driven some seven miles inland on the marshes of Louisiana by the hurricane of 1893. Mr. Wood contracted to deliver this schooner afloat in the Gulf of Mexico. To do this he excavated a canal, five miles long, through which he floated the schooner intact into the Gulf. He was successful in raising many sunken steamboats and other vessels in the Mississippi River and its tributaries, including the U. S. S. *Wynoka* which had sunk near the mouth of Red River, in water of such depth that it was thought she could not be raised.

Mr. Wood was the originator of many special devices for use in his particular line of operations, and was the patentee of an apparatus which has been successfully used in extinguishing fires on shipboard. He was also the first to use an electro-magnet for lifting steel and iron from beneath the water, having utilized this device to rescue a cargo of nails and wire which had sunk in deep water in front of New Orleans.

In 1886, Mr. Wood was married to Miss Ida Pettiss who, with five children, survives him. He also leaves two brothers and one sister.

Although Mr. Wood was a man of the most quiet habits, amounting almost to reserve, he had a subtle magnetism which drew other men to him in the warmest of friendships. He will long be remembered with affectionate regard by a large number of loyal friends.

Mr. Wood was elected a Member of the American Society of Civil Engineers on October 7th, 1908. He was also a Past-President of the Louisiana Engineering Society.

* Memoir prepared by J. F. Coleman, M. Am. Soc. C. E.

IRVING HAWKINS, Assoc. M. Am. Soc. C. E.*

DIED MARCH 14TH, 1911.

Irving Hawkins was born on a farm in South Haven, Long Island, on December 14th, 1863. When he was fourteen years of age his father died, and he was obliged to leave school and assist on the farm. Later, he took up the trade of a brick mason. From his early youth Mr. Hawkins was a student, and though employed during the day, he spent his evenings with his books. He deeply regretted having to give up school, for it was his desire to become an engineer. A substantial dam on the old farm, built by his older brother, Mr. E. C. Hawkins, now Chief Engineer and General Manager of the Morgan-Guggenheim Copper River and Northwestern Railway, in Alaska, and himself, when he was fourteen years old, still stands to attest their bent for engineering.

It was not until 1891, however, that Mr. Hawkins really took up engineering as his life work, but from that time he constantly followed his profession with success. His first employment was as Chainman on irrigation work in Wyoming, but he soon forged ahead as Rodman, Computer, and Levelman on surveys and construction on other large irrigation works in Idaho and Colorado.

In January, 1895, he returned East and remained a year with Mr. Emmett F. Smith, a Civil Engineer, of Patchogue, L. I., who says of him: "A man of sterling worth and integrity—nothing for show or for self—pure gold."

From January, 1896, to the fall of 1898, Mr. Hawkins was employed with the New York State Corps of Engineers on canal and highway work. For the remainder of 1898 he was with the United States Deep Waterway Service, on the Champlain Division.

After spending several months in the office of J. V. Davies, M. Am. Soc. C. E., Mr. Hawkins, in the spring of 1899, took up railroad construction work in West Virginia, remaining in that section from June, 1899, to June, 1902. During this period he was entrusted with highly responsible work, and acquitted himself creditably.

In 1902 he entered the writer's service and remained with him until 1909, during which period he was constantly in responsible charge. Among other work entrusted to him, he is entitled to the credit for the remarkably close results in line and grade in building a 3 500-ft. tunnel, 48 in. finished diameter, and driven through water-bearing drift under air pressure. The small size of the locks, shaft, and tunnel made precise work difficult, yet the headings met within ½ in.

* Memoir prepared by Alexander Potter, Assoc. M. Am. Soc. C. E.

In whatever branch of engineering Mr. Hawkins was engaged, the same thoroughness characterized his work. He was at his best, however, on outside work.

In December, 1908, he went to Santo Domingo, in charge of a topographic survey for that country. He remained there until December, 1909, by which time revolution, disease, and flood had decimated his party, and had made living in that country so unendurable that he returned to New York.

Shortly after his return, Mr. Hawkins was married to Miss Jennie C. Overton, of Bayport, L. I.

The exposure in Santo Domingo, which had robbed him of his coworkers, tardily claimed him as a victim, for in the spring of 1910, after working for a very short time with the Astoria Light, Heat and Power Company, on Long Island, he suffered from a nervous breakdown, from which he never fully recovered. He had improved so much, however, by March, 1911, that his physicians were of the opinion that he could shortly take up his work again. On March 14th, however, while out for a walk along the railroad several miles above Kingston, N. Y., where he was stopping at the time, he stepped off the track to allow a freight train to pass and was caught by a flying express coming from the opposite direction, and almost instantly killed.

Careful, industrious, attentive to business, thoroughly trustworthy, and rigidly and ruggedly honest, Mr. Hawkins always commanded and retained the respect and confidence of those for whom he worked and of those who worked for him. His cardinal characteristics were unselfishness and rigid devotion to duty.

All through his work Mr. Hawkins felt and expressed deep gratification that it afforded opportunities to make living conditions easier, healthier, and better. Everything that tended toward the uplifting of humanity appealed to him, and because of that came his great love for his work. As an evidence of his humane interest, he was a member of the Red Cross Society, and also a member of the National Child Labor Committee.

He is survived by his widow, two brothers, E. C. Hawkins and Emmett S. Hawkins, and a sister, Mrs. D. F. Glover.

Mr. Hawkins was elected an Associate Member of the American Society of Civil Engineers on May 6th, 1908.

TAKEJIRO SHIMA, Assoc. M. Am. Soc. C. E.*

DIED DECEMBER 22D, 1910.

Takejiro Shima was born on November 23d, 1871, in Saga, in the north of Kyushu Island, Japan. From boyhood he was distinguished for his scholarly ability, and, in 1894, at the age of twenty-three, he was graduated from the Engineering College of the Tokyo Imperial University. In December of the same year, he enlisted as a volunteer in the Japanese Army. During the Japan-China War, in 1894-1895, he exhibited bravery and heroism, and, in March, 1897, was promoted to the rank of Sub-Lieutenant (in engineering).

On his return to civil life, Mr. Shima was engaged by the Nankai Railway Company, and for three years discharged his duties as Engineer with remarkable ability and thoroughness. In June, 1898, he was appointed Railway Engineer of the Imperial Government Railways of Japan.

In April, 1902, Mr. Shima commenced a three-year tour of inspection and study throughout Europe and America, and returned to Japan in April, 1905, with a vast store of present-day railway knowledge.

In October, 1906, he was engaged by the South Manchuria Railway Company, his principal work being the conversion of the Antung-Mukden Light Railway into a broad-gauge line. This was doubtless the company's most important and most difficult enterprise, and the fact that it is now rapidly nearing completion is due in no small measure to Mr. Shima's arduous efforts. From the day of the company's foundation, he was a most prominent figure, and played an important rôle in the construction work of the railway and the reconstruction of the lines from narrow to broad gauge. In November, 1910, the reconstruction work of the Antung-Chikwanshan line was completed, and, on December 11th, Mr. Shima left Dairen for Chikwanshan to inspect the newly constructed section of the broad-gauge line. One afternoon, a week later, while riding in a hand-car, to inspect the finished work between Shahochen and Antung, he met with an accident, which, notwithstanding the skill of the surgeons, caused his death, on December 22d, 1910.

Mr. Shima's meritorious service as Acting Superintendent of the Gas Department of the South Manchuria Railway Company also deserves special mention. Shortly before his death, he was awarded the Sixth Order of Merit of the Sacred Treasure by the Imperial Japanese Government, in recognition of services rendered to the State. He

* Memoir prepared by M. Numata, Secretary, South Manchuria Railway Company.

was a good linguist, speaking English, German, and Chinese as well as his native tongue. He was exceedingly modest, and was never known to boast of his profound scholarship. He had a great liking for outdoor sports and exercises, and was noted for his strength.

Though he passed away at the premature age of forty, with bright prospects, which unfortunately were never to be fulfilled, his merits, his amiability of disposition, and the deeds which he had achieved, will always be remembered by those who had the pleasure of knowing him.

Mr. Shima was elected an Associate Member of the American Society of Civil Engineers on March 4th, 1903.

BASIL HENRY LEATHER, Assoc. Am. Soc. C. E.*

DIED OCTOBER 31ST, 1911.

Basil Henry Leather was born in Rainhill, Lancashire, England, on April 12th, 1865. He was educated at the Huyton College School, near Liverpool, England, and subsequently studied chemistry for several years. From 1884 to 1890 he was engaged in various commercial offices in Liverpool and Birmingham, and in the latter year came to America.

From 1890 until the spring of 1902 he was with the Neuchatel Asphalt Company, serving as Superintendent of Construction for the last nine years of that period. In the spring of 1902 he formed the Union Construction and Waterproofing Company, of which he was President until his death.

He had charge of the paving of the New York and Brooklyn Terminals of the Brooklyn Bridge, and also the stations of the elevated railway connected therewith, and the flat floor system of the loop tracks around the Plaza in Brooklyn. He also had charge of the paving of the tracks and upper deck platforms of the Jersey City Terminal of the Pennsylvania Railroad, in Jersey City; the water-proofing and heavy concrete foundations of the Manhattan Railroad sub-stations, in New York City; the water-proofing of the manholes of the conduit system of the Manhattan Railroad, and also those of the Telephone Company, where subject to tidal pressure. The asphalt pavements of the flat floor system of the elevated structures on the Nostrand Avenue and Ralph Avenue sections of the Atlantic Avenue Improvement, in Brooklyn, N. Y., were constructed by him, and also two sub-surface power-stations of the Metropolitan Railroad at Forty-second Street and the North River, New York City.

The most severe and particular work accomplished by Mr. Leather was the water-proofing of the lowest sections of the East River Tunnels of the Pennsylvania Railroad Terminal Improvement, New York City, and this probably brought about the complications which caused his death. In this same improvement he also water-proofed the low sections of the Terminal excavations from Seventh to Tenth Avenues and from Thirty-first to Thirty-third Streets.

He was also engaged on much bridge and special work for the Central Railroad of New Jersey, the Delaware, Lackawanna and Western, and the New York, Ontario and Western Railroads. Mr. Leather also constructed battery-room floors for the Manhattan and

* Memoir prepared by the Secretary from information furnished by H. H. Knowles, Esq., and from papers on file at the House of the Society.

Metropolitan Railroads and the New York Edison Company, for which latter company he did much other special work during the last ten years.

He was married in 1889 to Margaret Cecilia Quigley, eldest daughter of the late Thomas Quigley, Esq., of Clonmore, County Waterford, Ireland, who, with four sons, survives him.

Mr. Leather was elected an Associate of the American Society of Civil Engineers on June 1st, 1904.

EARL EDWIN ERDMANN, Jun. Am. Soc. C. E.*

DIED MARCH 28TH, 1910.

Earl Edwin Erdmann was born at Big Suamico, Wis., on June 15th, 1883. His early education was received at the local schools and at the West Green Bay High School. In 1903 he entered the University of Wisconsin, where he remained for two years. In 1905 he entered the University of Washington, at Seattle, Wash., receiving the degree of B. S. in Civil Engineering from that institution in June, 1907.

Mr. Erdmann began his engineering work in the summer of 1905 on maintenance of way on the Chicago Division of the "Big Four" Railway. In the summer of 1906 he was employed as Instrumentman by the Seattle Electric Company, and as Draftsman and Topographer on location for the Oregon and Washington Railroad Company.

After his graduation in 1907, he was employed as Instrumentman in building the substructure of the Columbia River Bridge for the Chicago, Milwaukee and St. Paul Railway Company, of Washington, until September, 1908, when he was appointed Resident Engineer in charge of the construction of the falsework for the steel erection and trestle approaches for the same bridge, and also of the engineering work required in connection with the erection of the steel superstructure.

On the completion of the work on the Columbia River Bridge, Mr. Erdmann was appointed Inspector of Bridges for the Chicago, Milwaukee and Puget Sound Railway Company, covering the territory from Butte, Mont., to Seattle and Tacoma, Wash., which position he held until his final illness.

In 1905, Mr. Erdmann was married to Miss Iva L. Hill, who, with a son, survives him.

Mr. Erdmann was a man of great determination, a faithful worker, and gave promise of becoming an able engineer.

He was elected a Junior of the American Society of Civil Engineers on July 17th, 1909.

* Memoir prepared by R. H. Ober, M. Am. Soc. C. E.

JOSEPH JAMES FERRIER, Jun. Am. Soc. C. E.*

DIED OCTOBER 29TH, 1911.

Joseph James Ferrier was born at Brighton, Sussex, England, on November 26th, 1882. His preliminary education was obtained in the local schools, and he spent considerable time voluntarily on the London, Brighton, and South Coast Railway, under the supervision of his father, who was an Inspector on that road. In 1902 he entered the Lintott Engineering Works, at Horshaw, where he was engaged in foundry work and on agricultural machinery, etc.

In February, 1903, Mr. Ferrier came to the United States and entered the employ of the Mergenthaler Linotype Company, as a Timekeeper, Draftsman, etc. During this time he took a full course in civil engineering with a correspondence school. In 1905 he was engaged as a draftsman on power-house design and details on the Port Morris and Yonkers stations of the New York Central and Hudson River Railroad. He remained with this company until 1907 and had then been promoted to the position of Chief Draftsman, and acted as Assistant to the Engineer of the power-station. In 1907 he resigned this position to accept one with the Southern Pacific Company on the electrification of its lines in and near San Francisco, Cal. Here he was engaged in the design and details of two large power plants, and took a prominent position in the Steam and Mechanical Engineering Branches of the Oakland, Alameda, and Berkeley electrification. Subsequently, he was promoted to the position of Office Engineer, and in this capacity handled many details in connection with this work.

Mr. Ferrier's death occurred on the afternoon of Sunday, October 29th, 1911, at his home in Fruitvale, Cal., the immediate cause being over-exertion in physical culture exercises, which he had been in the habit of taking daily. The shock to his family and friends was intensified by the unexpectedness and the seeming irony of the tragedy, for none was more vital, hopeful, or full of promise than he.

He was an exceedingly able, conscientious, and thorough engineer, and therefore was a very valuable assistant.

Mr. Ferrier was a member of the Masonic Order, the Foresters, and the Eastern Star, and a Junior of the American Society of Mechanical Engineers. He was elected a Junior of the American Society of Civil Engineers on December 3d, 1907.

* Memoir prepared by the Secretary from information furnished by A. H. Babcock, Esq., and from papers on file at the House of the Society.

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